AISI S240-20

American Iron and Steel Institute

AISI STANDARD

North American Standard for Cold-Formed Steel Structural Framing

2020 Edition

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DISCLAIMER

The material contained herein has been developed by the American Iron and Steel Institute (AISI) Committee on Framing Standards. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel framing design and installation. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the *Commentary*.

With anticipated improvements in understanding of the behavior of cold-formed steel framing and the continuing development of new technology, this material will become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a design professional. Indeed, in many jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all liability arising therefrom.

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PREFACE

The American Iron and Steel Institute (AISI) Committee on Framing Standards has developed AISI S240, *North American Standard for Cold-Formed Steel Structural Framing*, to address requirements for construction with cold-formed steel structural framing that are common to prescriptive and engineered design. This Standard is intended for adoption and use in the United States, Canada and Mexico.

In this edition, the following changes and additions are made:

1. The relevant provisions of AISI S201-17, *North American Standard for Cold-Formed Steel Framing – Product Data*, are incorporated into this Standard. The AISI S201 Standard will no longer be maintained.
2. In Section A1, Scope and Applicability, Section A1.2.4 is added to define the governing standard if conflict exists between this Standard and the *applicable building code*.
3. The definition of repetitive framing is deleted from Section A2. Framing member spacing requirements are added to the applicable sections as required.
4. A user note is added in Section A3.1 to clarify the materials that are applicable for cold-formed steel framing. Materials applicable to steel trusses are clarified in Section A3.3.
5. All the referenced standards are updated to the newest edition in Section A6. Chapter references and notations are updated throughout the Standard to be consistent with AISI S100-16 with Supplement 2.
6. Table A5-5 is expanded to include tracks with the flange width designation of 150.
7. For members with depth-to-thickness ratios larger than 200 but less than 260, web crippling strength is permitted to be determined using the provisions given in Section B1.6.
8. Screw edge distance is revised to 1.5 times the screw nominal diameter in Section B1.5.1.3.1.
9. Provisions are added in Section B2.2.1 to allow strength increase by 15% for floor joist assemblies that meet the criteria listed in the section.
10. A user note is added in Section B3.2.4, which provides guidance on how to consider the second order effect in determining the member forces.
11. The stud bearing length upper limit is revised to 2.375 in. (60.3 mm) in Section B3.2.5.1.
12. In Section B5.3, Strap Braced Wall Design, for height-to-length ratios greater than 2:1, design provisions for considering stud end moment due to joint fixity are added (in Section B5.3.1).
13. Steel deck and panel applications are added in Section B1.2.4, Spacing of Framing Members, and Section B5.4.1, Cold-Formed Steel Sheathed Diaphragms. Relevant new terms are introduced in Section A1.2, Definitions.
14. Explanations are provided in *Commentary* Sections B5.2 and B5.3 on how to determine the chord stud member forces when considering the second-order effects.
15. Section C4.1.4, Overdriven Screws in Shear Walls and Diaphragms Sheathed With Structural Panels, is added.
16. Clarification is made in Section C4.2 for welded areas that require treatment to retain corrosion resistance.
17. The concept of *basic frame inspection (BFI)* is introduced in Chapter D in order to differentiate *inspection* tasks intended to be performed by the *authority having jurisdiction* from *inspection* tasks intended to be performed by the *quality assurance inspector* and to

clarify when these *inspection* tasks are required.

18. Sections D2.1 and D6.5.1 are revised to better coordinate with IBC provisions related to *quality control* and *quality assurance* of cold-formed steel *component assemblies*.
19. Table D6.10-5 is revised to clarify the system installation tasks under item A.
20. A new test standard AISI S921, *Test Standard for Determining the Strength and Serviceability of Cold-Formed Steel Truss Assemblies and Components,* was developed and referenced in this Standard. Accordingly, Test Methods for Truss Components and Assemblies, which was included in Appendix 2 of the previous edition of the Standard, are removed.

While not necessary, use of the more stringent requirements for *structural members* that are in this Standard for *nonstructural members* should be permitted, since these should demonstrate equivalent performance for the intended use to those specified in AISI S220, *North American Standard for Cold-Formed Steel Nonstructural Framing*.

The Committee acknowledges and is grateful for the contributions of the numerous engineers, researchers, producers and others who have contributed to the body of knowledge on the subjects. The Committee wishes to also express its appreciation for the support of the Canadian Sheet Steel Building Institute.

American Iron and Steel Institute
November 2020

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SYMBOLS

Symbol	Definition	Section
A_c	Gross cross-sectional area of *chord member*, in square inches (mm^2)	B5.2.5, B5.4.2.4.1
A_g	Gross area of the clip angle *bearing stiffener*	B2.5.1
a	Distance between center line of braces	B2.6, B4.5
a	Wall aspect ratio	B5.2.2.3.2.1
B_1	Multiplier to account for P-δ effects	B3.2.4
B_2	Multiplier to account for P-Δ effects	B3.2.4
b	Length of the *shear wall*, in inches (mm)	B5.2.5
b	*Diaphragm* depth parallel to direction of *load*, in inches (mm)	B5.4.2.4.1
b	Effective width determined in accordance with Section B2.1 of AISI S100 [CSA S136] with $f=F_y$, k=4 and $w=W_{min}$	E4.5.1
C	Web crippling coefficient	B1.6, B3.2.5.1
C	Uplift anchorage force contributed from each level, lbs [kN]	B5.2.4.2.2
C	Uplift anchorage force, lbs [kN]	B5.2.4.2.3
C	Boundary chord force (tension/compression), lbs [kN]	B5.3.4.2
C_a	Shear resistance adjustment factor from Table B5.2.2.2-1. For intermediate values of opening height ratio and percentages of full-height sheathing, the shear resistance adjustment factors are permitted to be determined by interpolation.	B5.2.2.2, B5.2.4.1.2, B5.2.4.2.3
C_b	Bending coefficient dependent on moment gradient	E4.4.2.2
C_h	*Web* slenderness coefficient	B1.6, B3.2.5.1
C_m	End moment coefficient in interaction formula	E4.4.2.1
C_N	Bearing length coefficient	B1.6, B3.2.5.1
C_R	Inside bend radius coefficient	B1.6, B3.2.5.1
c	Length of cope	E4.6.2.1
D	Dead l*oad*	B1.2.2.4, B5.2.2.3.3
d	Depth of *C-shape* section	B2.6, B4.5
d_c	Depth of cope	E4.6.2.1
d_h	Depth of hole	B1.6

SYMBOLS

Symbol	Definition	Section
E_s	Modulus of elasticity of steel = 29,500,000 psi (203,000 MPa)	B5.2.5, B5.4.2.4.1
e	Design end or slip gap (distance between *stud web* at end of *stud* and *track web*)	B3.2.5.2.2
e	Eccentricity of compression force with respect to the centroid of the full section of the *web member*	E4.4.4.1
e_h	Clean distance between web hole and edge of bearing	B1.6
EI_w	Sheathing bending rigidity	1.3
F	Concentrated design *load* [*factored load*]	B2.6, B4.5
F	Furring channels	A5.3.1
F	*w*a	B2.6, B4.5
F_c	Critical buckling stress	E4.4.2.2
F_m	Mean value of the fabrication factor	B5.4.2.5
F_n	Nominal global flexural stress	B1.2.2.3, B2.2.1, B3.3.1.1, B2.3.3, B3.2.2, B3.3.2.5, B3.3.3.1.1, B3.3.4.1.1, B4.2.3, E4.4.2.2, E4.4.2.3
F_n	Nominal compressive stress	B2.3.2, B4.2.2, E4.4.2.3
F_y	*Yield strength* of clip angle	B2.5.1
F_y	Design *yield strength* of *track* material	B3.2.5.2.2
F_y	*Yield strength* used for design	B1.2.2.3, B2.2.1, B1.2.1.1, B2.3.3, B3.2.2, B3.3.1.1, B3.3.2.5, B3.3.3.1.1, B3.3.4.1.1, B4.2.2, B4.2.3, E4.4.2.2
F_y	Specified minimum *yield strength*	E4.5.1
F_y	Yield stress of steel sheet sheathing	B5.2.2.3.2.1
F_{uf}	Minimum tensile strength of framing materials	B5.2.2.3.2.1
F_{ush}	Tensile strength of steel sheet sheathing	B5.2.2.3.2.1
F_{ut}	Tensile strength of the *track*	B3.2.5.1
G	Shear modulus of sheathing material, in pounds per square	B5.2.5, B5.4.2.4.1

SYMBOLS

Symbol	Definition	Section
	inch (MPa)	
h	Depth of flat portion of *stud web* measured along plane of *web*	B1.6, B3.2.5.1
h	Height of the *shear wall*, ft (m)	B5.2.1.1, B5.2.1.2, B5.2.2.1, B5.2.4.2.2, B5.2.4.2.3
h	Wall height, in inches (mm)	B5.2.5
h	*Strap braced wall* height, feet [m]	B5.3.1, B5.3.4.2
h	Flat width of *web* of section being coped	E4.6.2.1
h_p	Height of a *wall pier*	B5.2.1.1
I_{min}	Moment of inertia determined with respect to an axis parallel to the *web* of the *chord member*	E4.6.2.1
KL	Effective length	B3.2.1, E4.4.2.1
K_t	Effective length factor for torsion	B3.2.1, E4.4.2.1, E4.4.2.2
K_x	Effective length factor for buckling about x-axis	B3.2.1, E4.4.2.1, E4.4.2.2
K_y	Effective length factor for buckling about y-axis	B3.2.1, E4.4.2.1, E4.4.2.2
k_ϕ	Rotational stiffness	B2.2.1, B2.3.2, B2.3.3, B3.2.1, B3.2.2, B4.2.2, B4.2.3, 1.1, 1.2, 1.3
$k_{\phi c}$	*Connection* rotational restraint	1.3
$k_{\phi w}$	Sheathing rotational restraint	1.3
L	Angle or L-*header*	A5.3.1
L	Live *load*	B1.2.2.4
L	Span length	B3.3.3.1.1
L	Length of strap braced wall	B5.3.1
L	Length of *Type I shear wall*, feet [m]	B5.2.4.1.1
L	Length of *Type I shear wall* including anchor offsets, feet [m]	B5.2.4.2.2
L	Length of *strap braced wall* including anchor offsets, feet [m]	B5.3.4.2
L	*Diaphragm* length perpendicular to direction of *load*, in	B5.4.2.4.1

SYMBOLS

Symbol	Definition	Section
	inches (mm)	
L	Unbraced length of the compression *web member*	E4.4.4.1
L	Length of *truss*	E6.1.1
L_{eff}	Average length between the last rows of fasteners of adjacent *truss* members in the *connection*	E4.5.1
L_h	Vertical leg dimension of angle	B3.3.3.1.1, B3.3.3.1.2
L_m	Distance between braces	B2.3.2, B3.2.1, B3.2.2.2, B4.2.2.2, B4.2.3.2
L_t	Unbraced length of compression member for torsion	B3.2.1, E4.4.2.1, E4.4.2.2
L_x	Unbraced length of compression member for bending about x-axis	B3.2.1, E4.4.2.1, E4.4.2.2
L_y	Unbraced length of compression member for bending about y-axis	B3.2.1, E4.4.2.1, E4.4.2.2
L_1, L_2	One-half joist spacing to the first and second sides respectively, as illustrated in Figure B8.2-1	1.3
M	*Required* flexural *strength*	B3.3.2.5
M_{ay}	*Available flexural strength* [*factored resistance*] based on Sections E2 and E3 of AISI S100 [CSA S136]. Only flexural and local buckling need to be considered.	E4.4.4.1
$M_{\ell t}$	Moment from first-order elastic analysis using *LRFD*, *LSD*, or *ASD load* combinations, as applicable, due to lateral translation of the structure only	B3.2.4
M_m	Mean value of the material factor	B5.4.2.5
M_n	*Nominal flexural strength* defined in Section C3.1 of AISI S100 [CSA S136]	B3.3.2.5
M_n	*Nominal flexural strength* [*resistance*] defined in Chapter F of AISI S100 [CSA S136]	B3.3.2.5
M_n	*Nominal* flexural *strength* [*resistance*]	E4.4.2.2
M_{ne}	*Nominal flexural strength* [*resistance*] for yielding and global buckling	B1.2.2.3, B2.2.1, B3.2.2, B3.3.1.1, B3.3.2.1, B3.3.2.5, B3.3.3.1.1, B3.3.4.1.1, B4.2.3, E4.4.2.2, E4.4.2.3
M_{ng}	Gravity *nominal* flexural *strength* [moment *resistance*]	B3.3.3.1.1, B3.3.3.1.2,

SYMBOLS

Symbol	Definition	Section
		B3.3.4.1.1
$M_{n\ell o}$	*Nominal* flexural *strength* [*resistance*] for local buckling	B3.3.3.1.1, B3.3.4.1.1
M_{nt}	Moment from first-order elastic analysis using LRFD, LSD or ASD load combinations, as applicable, with the structure restrained against lateral translation	B3.2.4
M_{nu}	*Nominal* uplift flexural *strength* [moment *resistance*]	B3.3.3.1.2
M_{nxo}	*Nominal* flexural *strength* [*resistance*] determined at $f = F_y$	E4.4.2.3
M_u	*Required* flexural *strength* [moment due to *factored loads*]	B3.3.2.5
M_x	*Required* flexural *strength*	E4.4.2.3
M_y	Member yield moment	B1.2.2.3, B2.2.1, B3.2.2, B3.3.1.1, B3.3.2.1, B3.3.2.5, B3.3.3.1.1, B3.3.4.1.1, B4.2.3, E4.4.2.2, E4.4.2.3
$\overline{M}$	*Required* flexural *strength* [moment of *factored loads*]	B3.2.4, E4.4.4.1
$\overline{M}_x$	*Required* flexural *strength* [moment of *factored loads*]	E4.4.2.3
m	Distance from shear center to mid-plane of *web*	B2.6, B4.5
N	*Stud* bearing length	B1.6, B3.2.5.1
n	Number of *chord* splices in *diaphragm* (considering both *diaphragm chords*)	B5.4.2.4.1
n	Number of connections in the assembly with the same tributary area	B5.4.2.5
P	*Required* web crippling *strength*	B3.3.2.5
P	*Required* compressive axial *strength*	E4.4.2.3
P_j	End two-*flange web* crippling capacity of the *floor joist*	B2.5.1
P_L	Required lateral force	B2.6, B4.5
P_n	*Nominal* web crippling *strength* [*resistance*]	B3.3.2.3
P_n	*Nominal* web crippling *strength* determined in accordance with Section B3.3.2.3	B3.3.2.5
P_n	*Nominal* web crippling *strength* [*resistance*] computed by Section B3.3.2.3	B3.3.2.5
P_n	*Nominal shear strength* [*resistance*] of screw connections within the effective strip width, w_e, on the steel sheet	B5.2.2.3.2.1

SYMBOLS

Symbol	Definition	Section
	sheathing	
P_a	*Available* axial *strength* [*resistance*] based on Chapter E of AISI S100 [CSA S136]. Only flexural and local buckling need to be considered.	E4.4.4.1
P_n	*Nominal* axial compressive *strength* [*resistance*]	E4.5
P_{ne}	*Nominal axial strength* [*resistance*] for overall buckling	B2.3.2, B4.2.2, E4.4.2.3
P_{no}	*Nominal* axial *strength* [*resistance*] determined at $f = F_y$	E4.4.2.3
P_{ndt}	*Nominal strength* [*resistance*] of a single *deflection track* subjected to transverse *loads* and connected to its support at a fastener spacing not greater than the *stud* spacing	B3.2.5.2.2
P_{nst}	*Nominal strength* [*resistance*] for the *stud*-to-*track connection* when subjected to transverse *loads*	B3.2.5.1, B3.2.5.2.1
P_{nst}	*Nominal* web crippling *strength* [*resistance*]	B3.2.5.2.1
P_t	Interior two-*flange web* crippling capacity of the *rim track*	B2.5.1
P_u	*Required* web crippling *strength* [compression force due to *factored loads*]	B3.3.2.5
P_y	Member axial yield strength	B2.3.2, B4.2.2, E4.4.2.3
$\overline{P}$	*Required* compressive axial *strength* [axial force of *factored loads*]	B3.2.4, E4.4.2.3, E4.4.4.1
R	*Seismic response modification coefficient*	A1.2.2
R	*Stud* inside bend radius	B3.2.5.1
R	Uplift factor	B3.3.3.1.2
R	*Required* concentrated load *strength*	E4.4.2.3
R	Modification factor	E4.4.4
R	Reduction factor	E4.6.2.1
$\overline{R}$	*Required* concentrated load *strength* [factored concentrated *load*]	E4.4.2.3
R_dR_o	Seismic force modification factors	A1.2.2
R_n	*Nominal* interior one-*flange* loading *web* crippling *strength* [*resistance*]	E4.4.2.3
r	Radius of gyration of the full section about the minor axis	E4.4.4.1
S	Snow *load*	B1.2.2.4
S	*Stud* or *joist* framing member which has *lips*	A5.3.1

SYMBOLS

Symbol	Definition	Section
S	Center-to-center spacing of *studs*	B3.2.5.2.2
S_c	Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c	E4.4.2.2
S_e	Elastic section modulus of effective section calculated relative to extreme compression fiber at F_y	E4.4.2.2
S_n	*Nominal shear strength* [*resistance*]	B5.4.2.5
ST	Specified standard term load based on snow or occupancy loads acting alone or in combination	B5.2.2.3.3
s	Maximum fastener spacing at panel edges, in inches (mm)	B5.2.5, B5.4.2.4.1
s	Screw spacing on the panel edges	B5.2.2.3.2.1
T	*Track* section	A5.3.1
t	Design thickness of steel sheet sheathing	B5.2.2.3.2.1
t	*Stud design thickness*	B3.2.5.1
t	*Design thickness*	B1.6, B3.3.3.1.2
t	*Design thickness* of *gusset plate*	E4.5.1
t	*Design thickness* of section being coped	E4.6.2.1
t_c	*Design thickness* of the *C-shape* section	B3.3.2.3
t_f	Minimum design thicknesses of framing members	B5.2.2.3.2.1
t_{sh}	Design thickness of steel sheet sheathing	B5.2.2.3.2.1
t_t	*Track design thickness*	B3.2.5.1, B3.2.5.2.2
t_t	*Track design thickness*	B3.3.2.3, B3.2.5.1
$t_{sheathing}$	Nominal panel thickness, in inches (mm)	B5.2.5, B5.4.2.4.1
t_{stud}	Framing designation thickness, in inches (mm)	B5.2.5, B5.4.2.4.1
U	Channel or *stud* framing section which does not have *lips*	A5.3.1
V	Shear force in *Type I shear wall*, lbs [kN]	B5.2.4.1.1, B5.2.4.1.2, B5.2.4.2.2
V	Shear force in *Type II shear wall*, lbs [kN]	B5.2.4.2.3
V	Total lateral *load* applied to the *shear wall*, in pounds (N)	B5.2.5
V	Shear force in *strap braced wall*, lbs [KN]	B5.3.4.2
V	Total lateral *load* applied to the *diaphragm,* in pounds (N)	B5.4.2.4.1
V_F	Coefficient of variation of fabrication factor	B5.4.2.5

SYMBOLS

Symbol	Definition	Section
V_M	Coefficient of variation of material factor	B5.4.2.5
V_n	*Nominal strength* [*resistance*] for shear	B5.2.1.2, B5.2.2.1, B5.2.2.2
V_P	Actual calculated coefficient of variation of the test results, without limit	B5.4.2.5
V_Q	Coefficient of variation of the load effect	B5.4.2.5
v_n	*Nominal strength* [*resistance*] per unit length as specified in Section B5.2.2.3, lbs./ft (kN/m)	B5.2.2.1, B5.2.2.2
v	Unit shear force, plf (kN/m)	B5.2.4.1.1, B5.2.4.1.2
v	Shear demand, in pounds per linear inch (N/mm) = V/b	B5.2.5
v	Shear demand, in pounds per linear inch (N/mm) = V/(2b)	B5.4.2.4.1
W	Wind *load*	B1.2.2.4
W_m	Lesser of the actual *gusset plate* width or Whitmore section	E4.5.1
w	Uniform *design load* [*factored load*]	B2.6, B4.5
w	Length of the *shear wall*, ft (m)	B5.2.1.1, B5.2.1.2, B5.2.2.1
w_e	Effective width	B5.2.2.3.2.1
w_p	Length of a *wall pier*	B5.2.1.1
w_{dt}	Effective *track* length	B3.2.5.2.2
w_{st}	$20\ t_t + 0.56\alpha$	B3.2.5.1
X_i	Distance between the "ith" *chord*-splice and the nearest support, in inches (mm)	B5.4.2.4.1
x	Distance from concentrated *load* to brace	B2.6, B4.5
α	Coefficient for conversion of units	B3.2.5.1, B3.2.5.2.2, B3.3.2.3
α	Parameter determined in accordance with Equation B3.3.2.3-1 or B3.3.2.3-2	B3.3.2.3
α	Ratio of the average *load* per fastener based on a non-uniform fastener pattern to the average *load* per fastener based on a uniform fastener pattern (= 1 for a uniformly fastened *diaphragm*)	B5.4.2.4.1
α, α_1, α_2	Variables	B5.2.2.3.2.1

SYMBOLS

Symbol	Definition	Section
β	Factor used on deflection calculation	B5.2.5, B5.4.2.4.1
β_o	Target reliability index	B5.4.2.5
β_1, β_2, β_3	Variables	B5.2.2.3.2.1
ρ	Factor used on deflection calculation	B5.2.5, B5.4.2.4.1
ρ	Variable	B5.2.2.3.2.1
δ	Calculated deflection, in inches (mm)	B5.2.5
δ_b	Deflection of blocked *diaphragm* determined in accordance with Eq. B5.4.2.4-1	B5.4.2.4.2, B5.4.2.4, B5.4.2.4.1
δ_{ub}	Deflection of an unblocked *diaphragm*	B5.4.2.4.2
δ_v	Vertical deformation of anchorage/attachment details, in inches (mm)	B5.2.5
θ	Angle between plane of web and plane of bearing	B1.6
ΣL_i	Sum of lengths of *Type II shear wall segments,* ft (m)	B5.2.2.2, B5.2.4.1.2, B5.2.4.2.3
Δ_{ci}	Deformation value associated with "ith" *chord* splice, in inches (mm)	B5.4.2.4.1
ω_1, ω_2, ω_3, ω_4	Factors used in deflection calculation	B5.2.5, B5.4.2.4.1
ϕ	*Resistance factor*	B3.2.5.1, B3.2.5.2.1, B3.2.5.2.2, B3.3.2.3, B3.3.2.5, B3.3.3.1.1, B3.3.3.1.2, B3.3.4.1.1, B5.2.3, E4.4.2.3
ϕ_c	*Resistance factor*	B2.5.1, E4.5.1
ϕ_v	*Resistance factor*	B5.2.3, B5.4.2.3
ϕ_w	*Resistance factor* for *web* crippling	B1.6
Ω	*Safety factor*	B3.2.5.1, B3.2.5.2.1, B3.2.5.2.2, B3.3.2.3, B3.3.2.5, B3.3.3.1.1, B3.3.3.1.2, B3.3.4.1.1, B5.2.3, E4.4.2.3

SYMBOLS

Symbol	Definition	Section
Ω_c	*Safety factor*	B2.5.1, E4.5.1
Ω_v	*Safety factor*	B5.2.3
Ω_w	*Safety factor* for *web* crippling	B1.6

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NORTH AMERICAN STANDARD
FOR COLD-FORMED STEEL STRUCTURAL FRAMING

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NORTH AMERICAN STANDARD
FOR COLD-FORMED STEEL STRUCTURAL FRAMING

A. GENERAL

A1 Scope and Applicability

A1.1 Scope

This Standard applies to the design, manufacture, installation, and quality of *structural members* and *connections* utilized in *cold-formed steel light-frame construction* applications.

A1.2 Applicability

A1.2.1 The design and installation of *cold-formed steel* framing for the following systems shall be in accordance with AISI S100 [CSA S136] and this Standard:

(a) floor and roof systems in buildings,
(b) structural walls in buildings,
(c) *shear walls*, *strap braced walls* and *diaphragms* to resist in-plane lateral *loads*, and
(d) *trusses* for *load*-carrying purposes in buildings.

A1.2.2 *Cold-formed steel structural members* and *connections* in *seismic force-resisting systems* and *diaphragms* shall be designed in accordance with the additional provisions of AISI S400 in the following cases:

(a) In the United States and Mexico, in *seismic design categories (SDC)* D, E, or F, or wherever the *seismic response modification coefficient*, R, used to determine the seismic design forces is taken other than 3.
(b) In Canada, where the design spectral response acceleration S(0.2) as specified in the NBCC is greater than 0.12 and the seismic force modification factors, R_dR_o, used to determine the seismic design forces, are taken as greater than or equal to 1.56.

A1.2.3 *Cold-formed steel* framing for floor and roof systems, and structural walls, as listed in items A1.2.1(a) and A1.2.1(b), are also permitted to be designed solely in accordance with AISI S100 [CSA S136].

A1.2.4 This Standard shall govern over other standards, including those referenced in this Standard, in matters pertaining to elements falling within the scope of this Standard, as defined in Section A1.1. Where conflicts between this Standard and the *applicable building code* occur, the requirements of the *applicable building code* shall govern. In areas without an *applicable building code*, this Standard defines the minimum acceptable standards for elements falling within the scope of this Standard, as defined in Section A1.1.

A1.2.5 This Standard does not preclude the use of other *approved* materials, assemblies, structures or designs of equivalent performance.

A1.2.6 This Standard includes Chapters A through F and Appendix 1 in their entirety.

A2 Definitions

Where the following terms appear in this Standard in italics, they shall have the meaning herein indicated. Terms included in square brackets shall be specific to *LSD* terminology. Where a country is indicated in square brackets following the definition, the definition shall apply only in the country indicated. Terms not defined in Section A2 shall have the ordinary accepted meaning for the intended context.

Adjusted Shear Resistance. In *Type II shear walls*, the unadjusted shear resistance multiplied by the shear resistance adjustment factor.

Allowable Strength. *Nominal strength* divided by the *safety factor* R_n/Ω. [United States and Mexico]

Applicable Building Code. The building code under which the building is designed.

Approved. Acceptable to the *authority having jurisdiction*.

ASD (*Allowable Strength Design*). Method of proportioning *structural components* such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the *ASD load combinations*. [United States and Mexico]

ASD Load Combination. *Load* combination in the *applicable building code* intended for *allowable strength design* (allowable stress design). [United States and Mexico]

Authority Having Jurisdiction. An organization, political subdivision, office, or individual charged with the responsibility of administering and enforcing the provisions of the *applicable building code*.

Available Strength. *Design strength* or *allowable strength*, as appropriate. [United States and Mexico]

Base Steel Thickness. The thickness of bare steel exclusive of all coatings.

Basic Frame Inspection. Essential monitoring and *inspection* tasks to ensure that the material provided and work performed by the *component manufacturer* and *installer* comply with the requirements of the *applicable building code* and *approved construction documents*. *Basic frame inspection* excludes those tasks designated "special inspection" by the *applicable building code*.

Bearing Stiffener. Additional material that is attached to the *web* to strengthen the member against *web* crippling. Also called a *web* stiffener.

BFI. See *basic frame inspection*.

Blocking. C-shaped member, break shape, flat *strap* material, or component assemblies attached to *structural members*, flat *strap* or sheathing panels to transfer shear forces or stabilize members.

Blocking, Panel. Blocking that transmits shear between the panels of a *shear wall* or *diaphragm*.

Blocking, Stud. Blocking that provides torsional restraint to the *studs* in a *shear wall.*

Bracing. Structural elements that are installed to provide restraint or support (or both) to other *structural members* or *nonstructural members* so that the complete assembly forms a stable structure.

Ceiling Joist. A horizontal *structural member* that supports ceiling components and which may be subject to attic *loads*.

Chord. Member of a *shear wall*, *strap braced wall* or *diaphragm* that forms the perimeter, interior opening, discontinuity or re-entrant corner.

Chord Member. A *structural member* that forms the top or bottom component of a *truss.*

Chord Splice. The *connection* region between two *truss chord members* where there is no change in slope.

Chord Stud. Axial *load*-bearing *studs* located at the ends of *Type I shear walls* or *Type II shear wall segments* or *strap braced walls*.

Clip Angle. An L-shaped short piece of steel typically used for *connections*.

Cold-Formed Sheet Steel. Sheet steel or strip steel that is manufactured by (1) press braking blanks sheared from sheets or cut length of coils or plates, or by (2) continuous roll forming of cold- or hot-rolled coils of sheet steel; both forming operations are performed at ambient room temperature, that is, without any addition of heat such as would be required for hot forming.

Cold-Formed Steel. See *Cold-Formed Sheet Steel*.

Collector. Also known as a drag strut, a member parallel to the applied *load* that serves to transfer forces between *diaphragms* and members of the *lateral force-resisting system* or distributes forces within the *diaphragm*.

Component. See *Structural Component*.

Component Assembly. A fabricated assemblage which consists primarily of *cold-formed steel structural members* that is manufactured by the *component manufacturer.*

Component Manufacturer. The individual or organization responsible for the manufacturing of *component assemblies* for the project. Also referred to as *truss* manufacturer on projects involving *trusses*, but hereinafter will be referred to as *component manufacturer*.

Composite Steel Deck-Slab. A system comprised of structural concrete placed over *composite steel floor deck*, in which the *steel deck* acts as positive bending reinforcement for the slab during the service life of the structure.

Composite Steel Floor Deck. A specific *steel deck* profile used as a form to create a structural concrete slab with the *steel deck* as moment reinforcement. The *steel deck* has embossments, interlocking profile geometry, or other horizontal shear resistance devices to develop mechanical bond between the *steel deck* and concrete so the slab compositely resists vertical and *diaphragm* shear loads. Prior to composite action, the *steel deck* acts as a form deck or work platform.

Connection. Combination of structural elements and joints used to transmit forces between two or more members.

Connector. A device used to transmit forces between *cold-formed steel structural members*, or between a *cold-formed steel structural member* and another structural element.

Contractor. *Owner* of the building, or the person that contracts with the *owner*, who constructs or manages the construction of the building in accordance with the *construction documents*. Also referred to as *owner*'s representative for construction, but hereinafter will be referred to as contractor.

Construction Documents. Written, graphic and pictorial documents prepared or assembled for describing the design (including the structural system), location and physical characteristics of the elements of a building necessary to obtain a building permit and construct a building.

Cripple Stud. A *stud* that is placed between a *header* and a window or door head *track*, a *header* and wall top *track*, or a windowsill and a bottom *track* to provide a backing to attach finishing and sheathing material.

C-Shape. A *cold-formed steel* shape used for *structural members* and *nonstructural members* consisting of a *web*, two (2) *flanges* and two (2) *lips* (*edge stiffeners*).

Curtain Wall. A wall that transfers transverse (out-of-plane) *loads* and is limited to a superimposed vertical *load*, exclusive of sheathing materials, of not more than 100 lb/ft (1.46 kN/m), or a superimposed vertical *load* of not more than 200 lbs (0.890 kN).

Deflection Track. A *track* manufactured with extended *flanges* and used at the top of a wall to provide for vertical movement of the structure, independent of the wall *stud*.

Design Load. Applied *load* determined in accordance with either *LRFD load combinations* or *ASD* *load combinations*, whichever is applicable. [United States and Mexico]

Design Strength. *Resistance factor* multiplied by the *nominal strength*. [United States and Mexico]

Design Thickness. The steel thickness used in design.

Designation Thickness. The minimum *base steel thickness* expressed in *mils* and rounded to a whole number.

Diaphragm. Roof, floor or other membrane or *bracing* system that transfers in-plane forces to the *lateral force-resisting system*.

Eave Overhang. The horizontal projection of the roof measured from the outside face of exterior wall framing to the outside edge of the roof.

Edge Stiffener. See *Lip*.

Factored Load. Product of a *load factor* and the *nominal load* [*specified load*].

Factored Resistance. Product of *nominal resistance* and appropriate *resistance factor*. [Canada]

Fiberboard. A fibrous, homogeneous panel made from lignocellulosic fibers (usually wood or cane) and having a density of less than 31 pounds per cubic foot (pcf) (497 kg/m^3) but more than 10 pcf (160 kg/m^3).

Flange. For a *C-shape*, U-shape or *track*, that portion of the *structural member* or *nonstructural member* that is perpendicular to the *web*. For a furring channel, that portion of the *structural member* or *nonstructural member* that connects the *webs*.

Floor Joist. A horizontal *structural member* that supports floor *loads* and superimposed vertical *loads*.

Girt. Horizontal *structural member* that supports wall panels and is primarily subjected to bending under horizontal *loads*, such as wind *load*.

Grade. The designation of the minimum *yield strength*.

Gusset Plate. A *structural member* used to facilitate the *connection* of *truss chord* or *web members* at a *heel*, *ridge*, other pitch break, or *panel point*.

Hat-Shape. A singly symmetric shape consisting of at least two vertical *webs* and a horizontal stiffened *flange* which is used as a *chord member* in a *truss*.

Header. A horizontal *structural member* used over floor, roof or wall openings to transfer *loads* around the opening to supporting *structural members*.

Heel. The *connection* region between the top and bottom *truss chords* of a non-parallel *chord truss*.

Hold-Down (Tie-Down). A device used to resist overturning forces in a *shear wall* or *strap braced wall*, or uplift forces in a *cold-formed steel structural member*.

Inspection. When used in conjunction with *quality control*, *basic frame inspection* and *quality assurance*, it shall mean the systematic examination and review of the work for compliance with the appropriate documents, with appropriate subsequent documentation.

Installation Drawings. Drawings that show the location and installation of the *cold-formed steel* structural framing. Also referred to as *truss* placement diagram for *truss* construction.

Installer. Party responsible for the installation of *cold-formed steel light-frame construction*.

Jack Stud. A *stud* that does not span the full height of the wall and provides bearing for *headers*.

Joist. A *structural member* primarily used in floor and ceiling framing.

King Stud. A *stud*, adjacent to a *jack stud,* that spans the full height of the wall and supports vertical and lateral *loads*.

Lateral Force-Resisting System. The structural elements and *connections* required to resist racking and overturning due to wind forces or seismic forces, or other predominantly horizontal forces, or combination thereof, imposed upon the structure in accordance with the *applicable* *building code*.

Light-Frame Construction. Construction where the vertical and horizontal structural elements are primarily formed by a system of repetitive *cold-formed steel* or wood framing members.

Limit States. Those conditions in which a *structural member* ceases to fulfill the function for which it was designed. Those states concerning safety are called the ultimate limit states. The ultimate limit state for strength is the maximum *load*-carrying capacity. Limit states that restrict the intended use of a member for reasons other than safety, such as deflection and vibration, are called serviceability limit states. [Canada]

Lip. That part of a *structural* or *nonstructural member* that extends from the *flange* as a stiffening element.

Load. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load Effect. Forces, stresses, and deformations produced in a *structural component* by the applied *loads*.

Load Factor. Factor that accounts for deviations of the actual *load* from the *nominal load*, for uncertainties in the analysis that transforms the *load* into a *load effect*, and for the probability that more than one extreme *load* will occur simultaneously. [United States and Mexico]

LRFD (Load and Resistance Factor Design). Method of proportioning *structural components* such that the *design strength* equals or exceeds the *required strength* of the *component* under the action of the *LRFD load combinations*. [United States and Mexico]

LRFD Load Combination. Load combination in the *applicable building code* intended for strength design (*Load and Resistance Factor Design*). [United States and Mexico]

LSD (Limit States Design). Method of proportioning *structural components* (members, connectors, connecting elements and assemblages) such that no applicable *limit state* is exceeded when the structure is subjected to all appropriate *load* combinations. [Canada]

Mean Roof Height. The average of the roof eave height and the height to the highest point on the roof surface, except that eave height shall be used for roof angles less than or equal to 10 degrees (0.18 rad).

Mil. A unit of measurement equal to 1/1000 inch.

Multiple Span. The *span* made by a continuous member having intermediate supports.

Nominal Load. Magnitude of the *load* specified by the *applicable building code*. [United States and Mexico]

Nominal Resistance (*Resistance*). Capacity of a structure or *component* to resist the effects of *loads*, determined in accordance with this Standard using specified material strengths and dimensions. [Canada]

Nominal Strength. Strength of a structure or *component* (without the *resistance factor* or *safety factor* applied) to resist the *load effects*, as determined in accordance with this Standard. [United States and Mexico]

Non-Composite Steel Floor Deck. *Steel deck* used as a stay-in-place form for structural concrete that can be designed to resist superimposed loads in a non-composite manner.

Nonstructural Member. A framing member in a steel-framed system that is not a part of the gravity *load*-resisting system, *lateral force-resisting system* or building envelope.

Owner. The individual or entity organizing and financing the design and construction of the project.

Panel Point. The *connection* region between a *web member* and *chord member* in a *truss*.

Pitch Break. The *connection* region between two *truss chord members* where there is a change in slope, excluding the *heel*.

Plans. Also referred to as construction drawings. Drawings prepared by the building designer for the *owner* of the project. These drawings include but are not limited to floor plans, framing plans, elevations, sections, details, and schedules as necessary to define the desired construction.

Plan Aspect Ratio. The ratio of the length (longer dimension) to the width (shorter dimension) of the building.

Profiled Steel Panel. Product formed from steel coils into fluted profiles with top and bottom *flanges* connected by *web* members and having a singular or a repeating pattern.

Punchout. A hole made during the manufacturing process in the *web* of a steel framing member.

Purlin. Horizontal *structural member* that supports roof deck and is primarily subjected to bending under vertical *loads* such as snow, wind, or dead *loads*.

QA. See *quality assurance*.

QC. See *quality control*.

Quality Assurance. Special monitoring and *inspection* tasks to ensure that the material provided and work performed by the *component manufacturer* and *installer* meet the requirements of the *approved construction documents* and referenced standards. *Quality assurance* includes those tasks designated "special inspection" by the *applicable building code*.

Quality Assurance Inspector. Individual or agency designated to provide *quality assurance inspection* for the work being performed.

Quality Control. Controls and *inspections* implemented by the *component manufacturer* or *installer* to confirm that the material provided and work performed meet the requirements of the *approved construction documents* and referenced standards.

Quality Control Inspector. Individual or agency designated to perform *quality control inspection* tasks for the work being performed.

Quality Control Program. Program in which the *component manufacturer* or *installer*, as applicable, maintains detailed assembly or installation and *inspection* procedures to ensure conformance with the *approved installation drawings*, *plans*, *specifications* and referenced standards.

Rake Overhang. The horizontal projection of the roof measured from the outside face of a gable endwall to the outside edge of the roof.

Rational Engineering Analysis. Analysis based on theory that is appropriate for the situation, any relevant test data, if available, and sound engineering judgment.

Registered Design Professional. Architect or engineer who is licensed to practice their respective design profession as defined by the legal requirements of the jurisdiction in which the building is to be constructed.

Required Strength. Forces, stresses, and deformations produced in a *structural component*, determined by either structural analysis, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by this Standard. [United States and Mexico]

Resistance Factor (ϕ). Factor that accounts for unavoidable deviations of the actual strength from the *nominal strength* [*nominal resistance*] and for the manner and consequences of failure.

Ridge. The horizontal line formed by the joining of the top edges of two upward sloping roof surfaces.

Rim Track. A horizontal *structural member* that is connected to the end of a *floor joist*.

Risk Category. A categorization of buildings and other structures for determination of flood, wind, snow, ice, and earthquake *loads* based on the risk associated with unacceptable performance.

Roof Rafter. A horizontal or sloped, *structural member* that supports roof *loads*.

Safety Factor (Ω). Factor that accounts for the desired level of safety, including deviations of the actual *load* from the *nominal load* and uncertainties in the analysis that transforms the *load* into a *load effect*, in determining the *nominal strength* and for the manner and consequences of failure. [United States and Mexico]

Seismic Design Category (SDC). A classification assigned by the *applicable building code* to a structure based upon its *risk category* and the severity of the design earthquake ground motion at the site.

Seismic Force-Resisting System. That part of the structural system that has been selected in the design to provide energy dissipation and the required resistance to seismic forces prescribed in the *applicable building code*.

Shear Wall. A wall with *structural sheathing* attached to *cold-formed steel structural members* and designed to primarily resist lateral forces parallel to the wall.

Shop Drawings. Drawings for the production of individual *component assemblies* for the project.

Single Span. The *span* made by one continuous *structural member* without any intermediate supports.

Span. The clear horizontal distance between bearing supports.

Specifications. Written instructions, which, with the *plans*, define the materials, standards, design of the products, and workmanship expected on a construction project.

Specified Load. Magnitude of the *load* specified by the *applicable building code*, not including *load factors*. [Canada]

Static Load. A *load* or series of *loads* that are supported by or are applied to a structure so gradually that forces caused by change in momentum of the *load* and structural elements can be neglected and all parts of the system at any instant are essentially in equilibrium.

Steel Deck. *Profiled steel panels* installed on support framing in a roof or floor assembly, including *steel roof deck*, *non-composite steel floor deck*, and *composite steel floor deck*.

Steel Panel Roof System. A roof system comprised of *profiled steel panels* that act as the roof covering, roof membrane, and sheathing for the roof *diaphragm*.

Steel Roof Deck. *Steel deck* panels used in a structural manner as a base for constructing and supporting the roof covering.

Steel Sheet Sheathing. A panel of thin flat steel sheet.

Strap. Flat or coiled sheet steel material typically used for *bracing* or *blocking*, which transfers *loads* by tension or shear.

Strap Braced Wall. A wall with *strap bracing* attached to *cold-formed steel structural members* and designed to primarily resist lateral forces parallel to the wall.

Strap Bracing. Steel *straps* applied diagonally to form a vertical *truss* that forms part of the *lateral force-resisting system*.

Structural Component. Member, *connector*, connecting element or assemblage.

Structural Member. A framing member that resists *design loads* [*factored loads*], as required by the *applicable building code*, except when defined as a *nonstructural member*.

Structural Sheathing. Sheathing that is capable of distributing *loads*, *bracing* members, and providing additional stability that strengthens the assembly.

Stud. A vertical *structural member* or *nonstructural member* in a wall system or assembly.

Track. A *structural member* or *nonstructural member* consisting of only a *web* and two (2) *flanges*. *Track web* depth measurements are taken to the inside of the *flanges*.

Truss. A coplanar system of *structural members* joined together at their ends typically to construct a series of triangles that form a stable beam-like framework.

Truss Design Drawing. Written, graphic and pictorial depiction of an individual *truss*.

Truss Design Engineer. Person who is licensed to practice engineering as defined by the legal requirements of the jurisdiction in which the building is to be constructed and who supervises the preparation of the *truss design drawings*.

Truss Designer. Person responsible for the preparation of the *truss design drawings*.

Truss Manufacturer. An individual or organization engaged in the manufacturing of site-built or in-plant *trusses*.

Truss Member. A *chord member* or *web member* of a *truss*.

Type I Shear Wall. Wall designed to resist in-plane lateral forces that is fully sheathed and that is provided with *hold-downs* at each end of the wall segment.

Type II Shear Wall. Wall designed to resist in-plane lateral forces that is sheathed with *wood structural panels* or *steel sheet sheathing* that contains openings, but which has not been specifically designed and detailed for force transfer around wall openings. *Hold-downs* for *Type II shear walls* are only required at the ends of the wall.

Type II Shear Wall Segment. Section of *shear wall* (within a *Type II shear wall*) with full-height sheathing (i.e., with no openings) and which meets specific aspect ratio limits.

Wall Pier. A section of a *Type I shear wall* adjacent to an opening and equal in height to the opening, which is designed to resist lateral forces in the plane of the wall.

Web. That portion of a *structural member* or *nonstructural member* that connects the *flanges*.

Web Member. A *structural member* in a *truss* that is connected to the top and bottom *chord members*, but is not a *chord member*.

Wind Exposure. Wind exposure in accordance with the *applicable building code*.

Wood Structural Panel. A panel manufactured from veneers, wood strands or wafers or a combination of veneer and wood strands or wafers bonded together with waterproof synthetic resins or other suitable bonding systems.

Yield Strength. Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Z-Shape. A point-symmetric or non-symmetric section that is used as a *chord member* in a *truss*.

A3 Material

A3.1 In the United States and Mexico, *structural members* utilized in *cold-formed steel light-frame construction* shall be cold-formed to shape from sheet steel complying with the requirements of ASTM A1003/A1003M, subject to the following limitations:

(a) Type H (high ductility): No limitations.

(b) Type L (low ductility): Limited to *purlins*, *girts* and *curtain wall studs*. Additional limitations for *curtain wall studs* are provided in Section A3.2.1.1 of AISI S100 [CSA S136].

User Note
ASTM A1003 was developed to be inclusive of ASTM A653/A653M, A792/A792M, A875/A875M and A1063/A1063M standards. For more information see CFSEI TN G801.

Therefore, products furnished to these material standards meet the requirements of A1003/A1003M.

Reference:
CFSEI, TN G801-13, *ASTM A1003 – No Cause for Rejection*, CFSEI, 2013.

A3.2 In Canada, *structural members* shall be cold-formed to shape from sheet steel complying with the requirements of ASTM A653/A653M Type SS or ASTM A792/A792M Type SS.

A3.3 In lieu of the requirements in Section A3.1 and A3.2, materials utilized in steel *trusses* shall comply with Section E4.1.

A4 Corrosion Protection

A4.1 Protective Coatings

A4.1.1 In the United States and Mexico, *structural members* utilized in *cold-formed steel light-frame construction* shall have a protective coating as specified in Table A4-1, with CP 60 minimum.

Table A4-1
Coating Designations

Coating Classification	Coating Designator	Minimum Coating Requirements			
		Zinc Coated A oz/ft² (g/m²)	Zinc Iron B oz/ft² (g/m²)	55% Al-Zinc C oz/ft² (g/m²)	Zinc-5% D oz/ft² (g/m²)
Metallic Coated	CP 60	G60 [Z180]	A60 [ZF180]	AZ50 [AZM150]	GF30 [ZGF90]
	CP 90	G90 [Z275]	Not Applicable	AZ50 [AZM150]	GF45 [ZGF135]
Painted Metallic	PM	The metallic coated substrate shall meet the requirements of metallic coated. In addition, the paint film shall have a minimum thickness of 0.5 mil per side (primer plus topcoat) with a minimum primer thickness of 0.1 mil per side. E			

A Zinc-coated steel sheet as described in ASTM A653/A653M.
B Zinc-iron alloy-coated steel sheet as described in ASTM A653/A653M.
C 55% Aluminum-zinc alloy-coated steel sheet as described in ASTM A792/A792M.
D Zinc-5% aluminum alloy-coated steel sheet as described in ASTM A875/A875M.
E In accordance with the requirements of ASTM A1003/A1003M.

A4.1.2 In Canada, *structural members* utilized in *cold-formed steel light-frame construction* shall have a minimum metallic coating of G60 [Z180] complying with the requirements of ASTM A653/A653M or AZ50 [AZM150] complying with the requirements of ASTM A792/A792M.

A4.2 Additional corrosion protection shall not be required on edges of metallic-coated steel framing members, where shop or field cut, punched or drilled.

A4.3 Unless additional corrosion protection is provided, framing members shall be located within the building envelope and shielded from direct contact with moisture from the ground or the exterior climate.

A4.4 Dissimilar metals shall not be used in direct contact with *cold-formed steel* framing members unless *approved* for that application.

A4.5 *Cold-formed steel* framing members shall not be embedded in concrete unless *approved* for that application.

A4.6 Fasteners shall have a corrosion-resistant treatment, or be manufactured from material not susceptible to corrosion.

A5 Products

A5.1 Base Steel Thickness

A5.1.1 The material thickness of framing members, in their end-use, shall meet or exceed the minimum *base steel thickness* values given in the *approved construction documents*. In no case shall the minimum *base steel thickness* be less than 95% of the *design thickness*.

A5.1.2 In the United States and Mexico, standard thicknesses are listed in Table A5-1. Member thickness shall be referenced to the corresponding *designation thickness*.

Table A5-1
Standard Thicknesses for United States and Mexico

Designation Thickness	Minimum Base Steel Thickness		Design Thickness	
	(inch)	(mm)	(inch)	(mm)
33	0.0329	0.836	0.0346	0.879
43	0.0428	1.087	0.0451	1.146
54	0.0538	1.367	0.0566	1.438
68	0.0677	1.720	0.0713	1.811
97	0.0966	2.454	0.1017	2.583
118	0.1180	2.997	0.1242	3.155

A5.1.3 In Canada, *structural members* shall be cold-formed to shape from sheet steel with a minimum *base steel thickness* listed in Table A5-2. Member thickness shall be referenced to the corresponding *designation thickness*.

Table A5-2
Standard Thicknesses for Canada

Designation Thickness	Minimum Base Steel Thickness		Design Thickness	
	(inch)	(mm)	(inch)	(mm)
33	0.0329	0.836	0.0346	0.879
43	0.0428	1.087	0.0451	1.146
54	0.0538	1.367	0.0566	1.438
68	0.0677	1.720	0.0713	1.811
97	0.0966	2.454	0.1017	2.583

A5.2 Minimum Flange Width

Where intended for sheathing attachment, *C-shape* members shall have a minimum *flange* width of 1-1/4 inch (31.8 mm). For *track* members, the minimum *flange* width shall be 3/4 inch (19.1 mm).

A5.3 Product Designator

A5.3.1 A four-part product designator that identifies the size (both *web* depth and *flange* width), type, and thickness shall be used for reference to *structural members*. The product designator as described (i.e., based on U.S. Customary units) shall be used for either U.S. Customary or SI Metric units. The product designator shall consist of the following sequential codes:

(a) A three- or four-digit numeral indicating member *web* depth in 1/100 inch.

(b) A letter indicating member type, in accordance with the following:

S = *C-shape* (commonly used as a *stud* or *joist* framing member) which has *lips*

T = *Track* section
U = Channel section which does not have *lips*
F = Furring channel
L = Angle or L-*header*

(c) A three-digit numeral indicating *flange* width in 1/100 inch, followed by a dash.
(d) A two- or three-digit numeral indicating *designation thickness*.

A5.3.2 The material *grade* used in design shall be identified on the *construction documents* and when ordering the material.

A5.4 Manufacturing Tolerances

Structural members utilized in *cold-formed steel light-frame construction* shall comply with the manufacturing tolerances listed in Table A5-3, as illustrated in Figure A5-1. All measurements shall be taken not less than 1 ft (305 mm) from the end of the member.

Table A5-3
Manufacturing Tolerances for Structural Members

Dimension[1]	Item Checked	C-shapes, in. (mm)	Tracks, in. (mm)
A	Length	+3/32 (2.38)	+ 1/2 (12.7)
		-3/32 (2.38)	-1/4 (6.35)
B[2]	Web Depth	+1/32 (0.79)	+1/32 (0.79)
		-1/32 (0.79)	+1/8 (3.18)
C	Flare	+1/16 (1.59)	+0 (0)
	Overbend	-1/16 (1.59)	-3/32 (2.38)
D	Hole Center Width	+1/16 (1.59)	NA
		-1/16 (1.59)	NA
E	Hole Center Length	+1/4 (6.35)	NA
		-1/4 (6.35)	NA
F	Crown	+1/16 (1.59)	+1/16 (1.59)
		-1/16 (1.59)	-1/16 (1.59)
G[3]	Camber	1/8 per 10 ft (3.13 per 3 m)	1/32 per ft (2.60 per m) 1/2 max (12.7)
H[3]	Bow	1/8 per 10 ft (3.13 per 3 m)	1/32 per ft (2.60 per m)
			1/2 max (12.7)
I	Twist	1/32 per ft (2.60 per m)	1/32 per ft (2.60 per m)
		1/2 max (12.7)	1/2 max (12.7)
J	Flange Width	+1/8 (3.18)	+1/4 (6.35)
		-1/16 (1.59)	-1/16 (1.59)
K	Stiffening Lip Length	+1/8 (3.18)	NA
		-1/32 (0.79)	

[1] All measurements are taken not less than 1 ft (305 mm) from the end.
[2] Outside dimension for *C-shape*; inside for *track*.
[3] 1/8 inch per 10 feet represents L/960 maximum for overall camber and bow. Thus, a 20-foot-long member has 1/4 inch permissible maximum; a 5-foot-long member has 1/16-inch permissible maximum.

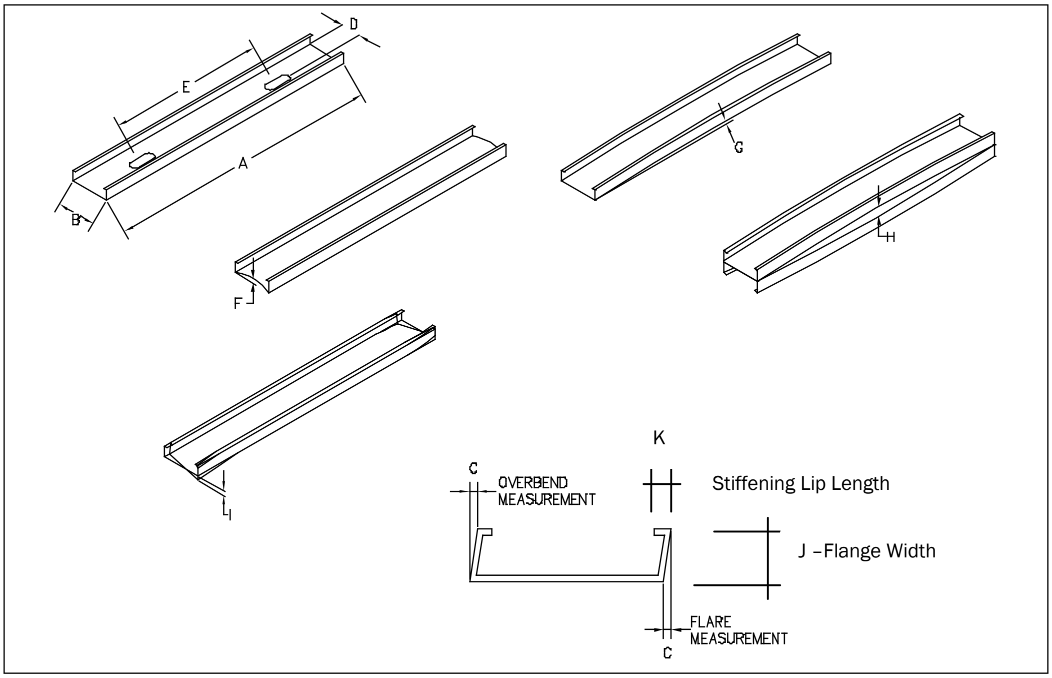

Figure A5-1 Manufacturing Tolerances for Structural Members

A5.5 Product Identification

Framing members used in *cold-formed steel light-frame construction* shall be identified in accordance with the requirements of this section.

A5.5.1 Identification of Groups of Like Members

A5.5.1.1 In the United States and Mexico, groups of like members shall be marked with a label, or a tag attached thereto. Marking shall include the roll-former's identification (name, logo, or initials), length, quantity, and roll-former's member designator including member depth, *flange* size, and minimum steel thickness in mils or inches exclusive of protective coating.

A5.5.1.2 In Canada, the identification of groups of like members is at the discretion of the manufacturer.

A5.5.2 Identification of Individual Framing Members

A5.5.2.1 In the United States and Mexico, in addition to the marking referenced in Section A5.5.1, individual framing members shall have a legible label, stencil, or embossment, at a maximum distance of 96 in. (2440 mm) on center, on the member, with the following minimum information:

(a) The rollformer's identification (i.e., name, logo, or initials).
(b) The minimum steel thickness, in mils or inches, exclusive of protective coating.
(c) The minimum *yield strength* in kips per square inch [megapascals].
(d) The appropriate protective coating designator in accordance with Section A4.

A5.5.2.2 In Canada, in addition to the marking referenced in Section A5.5.1, individual framing members shall have a legible label, stencil, or embossment, at a maximum distance of 96 in. (2440 mm) on center, on the member, with the following minimum information:

(a) The manufacturer's identification (name, logo, or initials); and
(b) The minimum steel thickness (in mils, inches or millimeters) exclusive of protective coatings.

A5.6 Standard Shapes

Standard shapes for *structural members*, as illustrated in Figure A5-2, are combinations of the basic dimensions listed in Tables A5-4 through A5-8, depending on the member type.

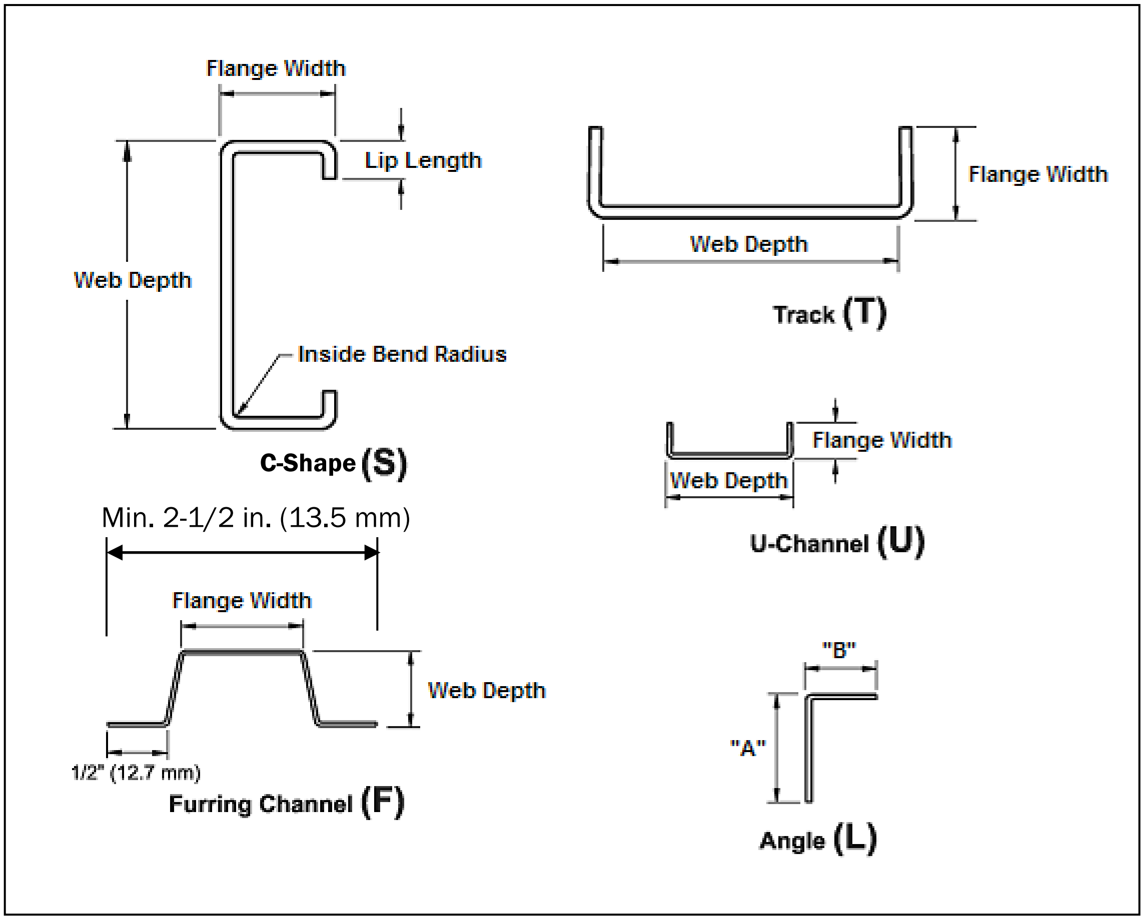

Figure A5-2 Standard Cold-Formed Steel Framing Member Types

Table A5-4
Standard Dimensions for C-Shapes (S)

Web Depth		
Depth Designation	Design Depth	
	(inch)	(mm)
162	1-5/8	41.3
250	2-1/2	63.5
350	3-1/2	88.9
362	3-5/8	92.1
400	4	102
550	5-1/2	140
600	6	152
800	8	203
1000	10	254
1200	12	305
1400	14	356

Flange Width		
Width Designation	Design Width	
	(inch)	(mm)
125	1-1/4	31.8
137	1-3/8	34.9
162	1-5/8	41.3
200	2	50.8
250	2-1/2	63.5
300	3	76.2
350	3-1/2	88.9

Notes: (1) Not all shapes are available in every standard thickness.
(2) Not all combinations of web depth and flange width are available.

Table A5-5
Standard Dimensions for Tracks (T)

Web Depth		
Depth Designation	Design Depth	
	(inch)	(mm)
162	1-5/8	41.3
250	2-1/2	63.5
350	3-1/2	88.9
362	3-5/8	92.1
400	4	102
550	5-1/2	140
600	6	152
800	8	203
1000	10	254
1200	12	305
1400	14	356

Flange Width		
Width Designation	Design Width	
	(inch)	(mm)
125	1-1/4	31.8
150	1-1/2	38.1
200	2	50.8
250	2-1/2	63.5
300	3	76.2

Notes: (1) Not all shapes are available in every standard thickness.
(2) Not all combinations of web depth and flange width are available.

Table A5-6
Standard Dimensions for U-Channels (U)

Web Depth		
Depth Designation	Design Depth	
	(inch)	(mm)
75	3/4	19.1
150	1-1/2	38.1
200	2	50.8
250	2-1/2	63.5

Flange Width		
Width Designation	Design Width	
	(inch)	(mm)
50	1/2	12.7
75	3/4	19.1

Notes: (1) Not all shapes are available in every standard thickness.
(2) Not all combinations of web depth and flange width are available.

Table A5-7
Standard Dimensions for Furring Channels (F)

Web Depth		
Depth Designation	Design Depth	
	(inch)	(mm)
87	7/8	22.2
150	1-1/2	38.1

Flange Width		
Width Designation	Design Width	
	(inch)	(mm)
125	1-1/4	31.8

Notes: (1) Not all shapes are available in every standard thickness.

Table A5-8
Standard Dimensions for Angles (L)

"A" Flange Width		
Depth Designation	Design Depth	
	(inch)	(mm)
62	5/8	15.9
87	7/8	22.2
137	1-3/8	34.9
150	1-1/2	38.1
200	2	50.8
300	3	76.2

"B" Flange Width		
Width Designation	Design Width	
	(inch)	(mm)
62	5/8	15.9
87	7/8	22.2
137	1-3/8	34.9
150	1-1/2	38.1
200	2	50.8
300	3	76.2

Note: (1) Not all shapes are available in every standard thickness.
(2) Not all combinations of "A" and "B" flange widths are available.

A5.7 Inside Bend Radius

Unless specified otherwise in the *construction documents*, the size of the inside bend radius used for design shall comply with the requirements shown in Table A5-9.

Table A5-9
Standard Design Inside Bend Radius

Designation Thickness	Inside Bend Radius (inch)	Inside Bend Radius (mm)
18	0.0843	2.141
27	0.0796	2.022
30	0.0781	1.984
33	0.0764	1.941
43	0.0712	1.808
54	0.0849	2.156
68	0.1069	2.715
97	0.1525	3.874
118	0.1863	4.732

A5.8 Lip Length

Unless specified otherwise in the *construction documents*, the *lip* length on a *C-shape* shall be related to the *flange* width as listed in Table A5-10.

Table A5-10
Standard Design Lip Length for C-Shapes (S)

Flange Width Designation	Flange Width		Design Lip Length	
	(inch)	(mm)	(inch)	(mm)
125	1-1/4	31.8	3/16	4.8
137	1-3/8	34.9	3/8	9.5
162	1-5/8	41.3	1/2	12.7
200	2	50.8	5/8	15.9
250	2-1/2	63.5	5/8	15.9
300	3	76.2	5/8	15.9
350	3-1/2	88.9	1	25.4

A5.9 Punchouts

Unless specified otherwise by the manufacturer, factory *punchouts* (perforations) shall comply with the following conditions:

(1) *Punchouts* shall be spaced along the centerline of the *web* of the framing member;
(2) *Punchouts* shall have a center-to-center spacing of not less than 24 inches (610 mm);
(3) *Punchouts* shall have a width not greater than half the member depth or 2-1/2 inches (63.5 mm), whichever is less;
(4) *Punchouts* shall have a length not exceeding 4-1/2 inches (114 mm); and
(5) The distance from the center of the last *punchout* to the end of the member shall not be less than 12 inches (305 mm), unless otherwise specified.

Any configuration or combination of holes that fits within the *punchout* width and length limitations is permitted.

A6 Reference Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

1. American Society of Civil Engineers, Reston, VA

   ASCE 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*

2. American Iron and Steel Institute, Washington, DC

   AISI S100-16 (2020) w/S2-20, *North American Specification for the Design of Cold-Formed Steel Structural Members* with Supplement 2

   AISI S202-20, *Code of Standard Practice for Cold-Formed Steel Structural Framing*

   AISI S310-20, *North American Standard for the Design of Profiled Steel Diaphragm Panels*

   AISI S901-17, *Test Standard for Determining the Rotational-Lateral Stiffness of Beam-to-Panel Assemblies*

   AISI S902-17, *Test Standard for Determining the Effective Area of Cold-Formed SteelCompression Members*

   AISI S903-17, *Test Standard for Determining the Uniform and Local Ductility of Carbon and Low-Alloy Steels*

   AISI S904-17, *Test Standard for Determining the Tensile and Shear Strengths of Steel Screws*

   AISI S905-17, *Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections*

   AISI S907-17, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by the Cantilever Test Method*

   AISI S909-17, *Test Standard for Determining the Web Crippling Strength of Cold-Formed Steel Flexural Members*

   AISI S910-17, *Test Standard for Determining the Distortional Buckling Strength of Cold-Formed Steel Hat-Shaped Compression Members*

   AISI S911-17, *Test Standard for Determining the Flexural Strength of Cold-Formed Steel Hat-Shaped Members*

   AISI S913-17, *Test Standard for Determining the Strength and Deformation Behavior of Hold-Downs Attached to Cold-Formed Steel Structural Framing*

   AISI S914-17, *Test Standard for Determining the Strength and Deformation Behavior of Joist Connectors Attached to Cold-Formed Steel Structural Framing*

   AISI S915-15, *Test Standard for Through-The-Web Punchout Cold-Formed Steel Wall Stud Bridging Connectors*

   AISI S916-15, *Test Standard for Cold-Formed Steel Framing—Nonstructural Interior Partition Walls With Gypsum Board*

   AISI S917-17, *Test Standard for Determining the Fastener-Sheathing Local Translational Stiffness of Sheathed Cold-Formed Steel Assemblies*

   AISI S918-17, *Test Standard for Determining the Fastener-Sheathing Rotational Stiffness of Sheathed Cold-Formed Steel Assemblies*

   AISI S919-17, *Test Standard for Determining the Flexural Strength and Stiffness of Cold-Formed Steel Nonstructural Members*

   AISI S921-19, *Test Standard for Determining the Strength and Serviceability of Cold-Formed Steel Truss Assemblies and Components*

3. ASTM International, West Conshohocken, PA

   ASTM A500/A500M-20, *Standard Specification for Cold-Formed Welded and Seamless Carbon*

Steel Structural Tubing in Rounds and Shapes

ASTM A653/A653M-19a, *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*

ASTM A792/A792M-10(2015), *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

ASTM A875/A875M-13, *Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process*

ASTM A1003/A1003M-15, *Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members*

ASTM C208-12 (2017)e2, *Standard Specification for Cellulosic Fiber Insulating Board*

ASTM C954–18, *Standard Specification for Steel Drill Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Steel Studs From 0.033 in. (0.84 mm) to 0.112 in. (2.84 mm) in Thickness*

ASTM C1002–18, *Standard Specification for Steel Self-Piercing Tapping Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Wood Studs or Steel Studs*

ASTM C1007-20, *Standard Specification for Installation of Load Bearing (Transverse and Axial) Steel Studs and Related Accessories*

ASTM C1396/C1396M–17, *Standard Specification for Gypsum Board*

ASTM C1513-18, *Standard Specification for Steel Tapping Screws for Cold-Formed Steel Framing Connections*

ASTM E455-19, *Standard Method for Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings*

ASTM E2126-19, *Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings*

4. American Welding Society, Miami, FL

   AWS B5.1:2013, *Specification for the Qualification of Welding Inspectors*

   AWS D1.3/D1.3M:2018, *Structural Welding Code-Sheet Steel*

5. CSA Group, Mississauga, Ontario, Canada

   CSA O325-16, *Construction Sheathing*

   CAN/CSA-S136-16 (2020) w/S2-20, *North American Specification for the Design of Cold-Formed Steel Structural Members* with Supplement 2

   CSA O121-17, *Douglas Fir Plywood*

   CSA O151-17, *Canadian Softwood Plywood*

   CSA O325-16, *Construction Sheathing*

   CSA O437-93 (R2011), *Standards on OSB and Waferboard*

6. Department of Commerce Voluntary Product Standard, administered by NIST, Gaithersburg, MD

   DOC PS 1-09, *Structural Plywood*

   DOC PS 2-10, *Performance Standard for Wood-Based Structural-Use Panels*

7. National Research Council of Canada, Ottawa, Ontario, Canada

   NBCC 2015, *National Building Code of Canada*, 2015 Edition

B. DESIGN

B1 General

The provisions in Section B1 shall be used in conjunction with the requirements in Sections B2 through B5, as applicable.

B1.1 Loads and Load Combinations

Buildings or other structures and all parts therein shall be designed in accordance with the *applicable building code* to support all *loads* that are expected during its life. In the absence of an *applicable building code*, the *loads*, forces, and combinations of *loads* shall be in accordance with accepted engineering practice for the location under consideration as specified by the applicable sections of ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* in the United States and Mexico, or the *National Building Code of Canada* in Canada.

B1.1.1 Application of Live Load Reduction on Wall Studs

For the purpose of calculating the design axial *load* on a wall using floor live *load* reduction requirements in accordance with the *applicable building code*, the tributary area of the wall shall be limited to the floor area assigned to the individual wall framing members.

B1.1.2 Application of Wind Loads on Wall Studs in the United States and Mexico

In the United States and Mexico, the design of the wall *studs* shall be in accordance with the applicable provisions of ASCE 7.

Exception: Where wall *studs* are spaced no greater than 24 inches (610 mm) on center, the design of the wall *studs* is permitted to be based on the following design wind loads:

(a) Combined bending and axial *load effect* based on Main Wind Force-Resisting System (MWFRS) wind *loads*.

(b) Bending *load effect* based on Components and Cladding (C&C) wind *loads*.

(c) Deflection limits based on 42% of Components and Cladding (C&C) wind *loads* with no axial *loads*.

B1.2 Design Basis

The proportioning, designing and detailing of *cold-formed steel light-frame lateral force-resisting systems*, *trusses*, *structural members*, *connections* and *connectors* shall be in accordance with AISI S100 [CSA S136], and the reference documents except as modified or supplemented by the requirements of this Standard.

B1.2.1 Floor Joists, Ceiling Joists and Roof Rafters

B1.2.1.1 *Floor joists*, *ceiling joists* and *roof rafters* shall be designed either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. Floor and roof assemblies using discretely braced design shall be designed neglecting the structural *bracing* and composite-action contribution of attached sheathing or deck. The discretely braced design requirements of the Standard shall be applied to assemblies where the sheathing or deck is not attached directly to *structural members*.

(b) Continuously Braced Design. Unless noted otherwise in Section B2 or B4, the continuously braced design requirements of this Standard shall be limited to assemblies where *structural sheathing* or *steel deck* is attached directly to *floor joists*, *ceiling joists* and *roof rafters* that comply with all of the following conditions:

(1) Maximum *web* depth = 14 inches (356 mm)
(2) Maximum *design thickness* = 0.1242 inches (3.155 mm)
(3) Minimum design *yield strength*, F_y = 33 ksi (230 MPa)
(4) Maximum design *yield strength*, F_y = 50 ksi (345 MPa)

B1.2.1.2 Where continuously braced design is used, the *construction documents* shall identify *structural sheathing* or *steel deck* as a structural element.

B1.2.1.3 A *web* with one or more holes shall be designed in accordance with AISI S100 [CSA S136], or reinforced in accordance with an *approved* design or as specified by a *registered design professional*.

B1.2.2 Wall Studs

B1.2.2.1 Wall *studs* shall be designed either on the basis of all steel design or on the basis of sheathing braced design, in accordance with the following:

(a) All Steel Design. Wall *stud* assemblies using all steel design shall be designed neglecting the structural *bracing* and composite-action contribution of the attached sheathings.

(b) Sheathing Braced Design. Wall *stud* assemblies using the sheathing braced design provisions of this Section shall have *structural sheathing* attached to both *flanges* of the wall *stud* or *structural sheathing* attached to the one *flange* and discrete bracing to the other *flange*. The *studs* shall be spaced no greater than 24 inches (610 mm) on center and be connected to the bottom and top *track* or other horizontal member(s) of the wall to provide lateral and torsional support to the wall *stud* in the plane of the wall. Wall *studs* with sheathing attached to both sides that is not identical shall be designed based on the assumption that the weaker of the two sheathings is attached to both sides.

User Note:
The contribution of other sheathing types to the structural bracing or composite-action of wall *stud* assemblies or use of alternative *stud* spacings are not precluded by this Standard when substantiated by *rational engineering analysis*.

B1.2.2.2 When sheathing braced design is used, the *construction documents* shall identify the *structural sheathing* as a structural element.

B1.2.2.3 For *curtain wall studs*, the combination of *structural sheathing* attached to one side of the wall *stud* and discrete bracing for the other *flange* is permitted. The spacing of discrete bracing shall be no greater than 8 ft (2.44 m) on center and the *studs* shall not be spaced greater than 24 inches (610 mm) on center. For design, the *nominal flexural strength* [*resistance*] shall be determined by Chapter F of AISI S100 [CSA S136]. When the compression *flange* has *structural sheathing* attached, the *available flexural strength* [*factored resistance*] is permitted to be the lesser of the strength determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136], where the contribution of the sheathing rotational stiffness, determined in accordance with Appendix 1, is permitted.

B1.2.2.4 In the United States and Mexico, when sheathing braced design is used, the wall *studs* shall also be evaluated without the sheathing *bracing* for the following *load* combination:

$$1.2D + (0.5L \text{ or } 0.2S) + 0.2W \qquad (Eq.\ B1.2.2\text{-}1)$$

where
D = Dead *load*
L = Live *load*
S = Snow *load*
W = Wind *load*

B1.2.2.5 In Canada, the provisions for sheathing braced design shall be in accordance with a theory, tests, or *rational engineering analysis* and shall comply with Chapters E and F of AISI S100 [CSA S136], as applicable.

B1.2.2.6 For *webs* with holes, the hole influence on member strength shall be considered in accordance with AISI S100 [CSA S136].

B1.2.3 In-Line Framing

B1.2.3.1 Each *joist*, *rafter*, *truss*, and structural wall *stud* (above or below) shall be aligned vertically in accordance with the limits depicted in Figure B1.2.3-1.

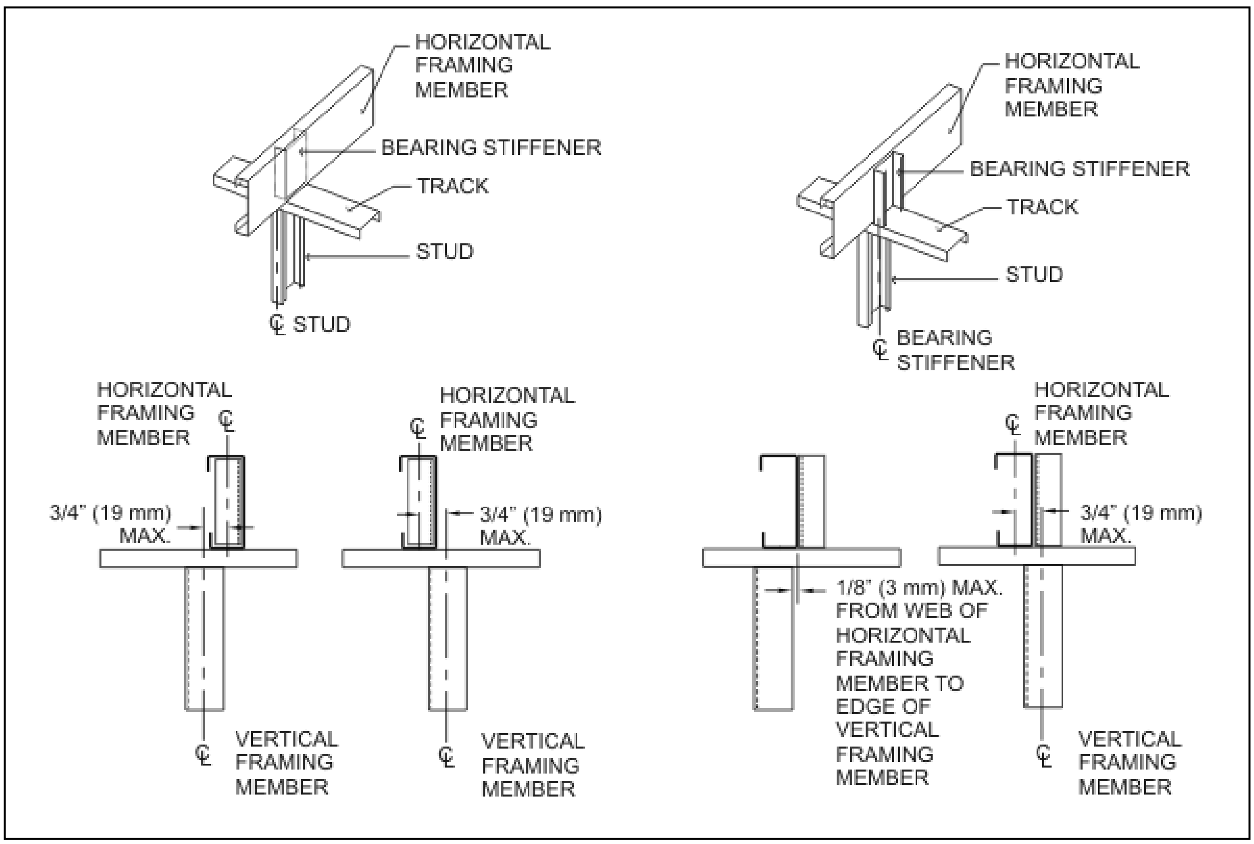

Figure B1.2.3-1 In-Line Framing

B1.2.3.2 The alignment tolerance shall not be required to be met when a structural *load* distribution member is specified in accordance with the *approved construction documents*.

B1.2.4 Spacing of Framing Members

B1.2.4.1 *Floor joist* and floor *truss* spacing shall be limited by the span capacity of the floor *structural sheathing*, *non-composite steel deck*, or *composite steel deck-slab*.

B1.2.4.2 Wall *stud* spacing shall be limited by the span capacity of the wall *structural sheathing*.

B1.2.4.3 The spacing of roof and ceiling framing members shall be limited by the span capacity of the ceiling sheathing, roof *structural sheathing*, *steel roof deck*, or *steel panel roof system*.

User Note:
For provisions in this Standard developed based on a specific member spacing criterion, the maximum member spacing is specified in the applicable section.

B1.2.5 Load Path

A continuous *load* path to the foundation shall be provided for the uplift, shear, and compression forces due to lateral wind forces, seismic forces, or other predominantly horizontal forces, or combinations thereof, imposed upon the structure in accordance with the *applicable building code*. Elements resisting forces contributed by multiple stories shall be designed for the sum of forces contributed by each story in conformance to *applicable building code*.

B1.2.6 Principles of Mechanics

Where *cold-formed steel structural members* and *connections* are not required to be designed in accordance with the additional provisions of Section A1.2.2, the provisions of this section are permitted for *lateral force-resisting systems*.

B1.2.6.1 The shear resistance of *shear walls*, *strap braced walls* and *diaphragms* is permitted to be determined by principles of mechanics using values of fastener strength, sheathing shear resistance, and *strap* strength, as applicable.

B1.2.6.2 When determined by the principles of mechanics, the *nominal strength* [*nominal resistance*] defines the maximum resistance that the *diaphragm*, *shear wall*, or *strap braced wall* is capable of developing.

B1.2.6.3 *Required strength* [effect of *factored loads*] shall be determined in accordance with the force requirements in the *applicable building code*.

B1.2.6.4 When determined by the principles of mechanics, values for systems defined in this Standard shall be scaled to the values in this Standard.

B1.3 Built-Up Section Design

Built-up sections shall be evaluated in accordance with Section I1 of AISI S100 [CSA S136] and the additional requirements of Sections B1.3.1 and B1.3.2, as applicable.

B1.3.1 For either all steel design or sheathing braced design, the *available strength* [*factored resistance*] of built-up sections shall be determined in accordance with Section I1.2 of AISI S100 [CSA S136].

Exception: Where a built-up axial *load* bearing section comprised of two *studs* oriented back-to-back forming an I-shaped cross-section is seated in a *track* in accordance with the requirements of Section C3.4.3 and the top and bottom end bearing detail of the *studs* consists of support by steel or concrete components with adequate strength and stiffness to preclude relative end slip of the two built-up *stud* sections, the compliance with the end *connection* provisions of AISI S100 Section I1.2(b) is not required.

B1.3.2 When the *connection* requirements of Section I1.2 of AISI S100 [CSA S136] or the exception permitted in B1.3.1 are not met, the *available strength* [*factored resistance*] of built-up sections shall be equal to the sum of the *available strengths* [*factored resistances*] of the individual members of the built-up cross-section.

B1.4 Properties of Sections

The properties of sections shall be full cross-section properties, except where use of a reduced cross-section or effective design width is required by AISI S100 [CSA S136].

B1.5 Connection Design

Connections using screws, welds, bolts, or power-actuated fasteners shall be designed in accordance with Chapter J of AISI S100 [CSA S136] and the additional requirements of this Standard. For *connections* using other fastener types, design values [*factored resistances*] shall be determined by testing in accordance with Section K2.1 of AISI S100 [CSA S136].

B1.5.1 Screw Connections

B1.5.1.1 Steel-to-Steel Screws

Screw fasteners for steel-to-steel *connections* shall be in compliance with ASTM C1513 or the *approved construction documents*.

B1.5.1.2 Sheathing Screws

Screw fasteners for *structural sheathing* to steel *connections* shall be in compliance with ASTM C1513 or the *approved construction documents*.

B1.5.1.3 Edge Distance, End Distance and Spacing

B1.5.1.3.1 For screw fasteners in steel-to-steel *connections* to be considered fully effective, the minimum edge or end distance shall be 1.5 times the nominal diameter.

B1.5.1.3.2 For screw fasteners in steel-to-steel *connections* to be considered fully effective, the minimum center-to-center spacing shall be 3 times the nominal diameter.

Exception: Where the center-to-center spacing of screw fasteners in steel-to-steel *connections* is less than 3 times the nominal diameter but greater than or equal to 2 times the nominal diameter, screw fasteners shall be considered 80 percent effective.

B1.5.1.4 Gypsum Board

Screw fasteners for gypsum board to steel *connections* shall be bugle head style in compliance with ASTM C954, ASTM C1002, or ASTM C1513, as applicable.

B1.5.2 Welded Connections

Welded *connections* shall be in accordance with Chapter J of AISI S100 [CSA S136] and AWS D1.3. The design capacity of welds shall be in accordance with Section J2 of AISI S100 [CSA S136].

B1.5.3 Bolts

Bolted *cold-formed steel connections* shall be designed and installed in accordance with Chapter J of AISI S100 [CSA S136].

B1.5.4 Power-Actuated Fasteners

Cold-formed steel connections using power-actuated fasteners shall be designed and installed in accordance with Chapter J of AISI S100 [CSA S136].

B1.5.5 Other Connectors

Other types of *connections* (e.g., rivet fasteners and clinch joining) shall be designed, fabricated and installed in accordance with the design requirements as set forth by the *approved construction documents* and the fastener manufacturer's requirements.

B1.5.6 Connection to Other Materials

Bolts, nails, anchor bolts or other fasteners used to connect *cold-formed steel* framing to wood, masonry, concrete or other steel components shall be designed and installed in accordance with the *applicable building code* or the *approved construction documents*.

B1.6 Web Crippling

The *available* web crippling *strength* [*factored resistance*] shall be determined in accordance with Sections G5 and G6 of AISI S100 [CSA S136], as applicable, unless otherwise specified. Where a *bearing stiffener* is used in accordance with the requirements of Section B2.5.1 or B4.4, web crippling does not need to be considered.

Exception: For *stud* or *joist webs* with depth-to-thickness ratio greater than 200 and less than 260 with one-flange loading condition, the *available web crippling strength* [*factored resistance*] is permitted to be 0.95 times the *nominal strength* [*resistance*] determined by Equation G5-1 of AISI S100 [CSA 136] with the coefficients and *safety* and *resistance factors* defined as follows:

$C = 4$

$C_R = 0.14$

$C_N = 0.35$

$C_h = 0.02$

$\Omega_w = 1.85$ (*ASD*)

$\phi_w = 0.80$ (*LRFD*)

$= 0.70$ (*LSD*)

The following conditions shall be satisfied:

$200 < h/t \le 260$

$N/t \le 210$

$N/h \le 2.0$

$\theta = 90°$

$e_h \geq 3.9\ d_h$

where

e_h = Clear distance between web hole and edge of bearing

Other variables are defined in Section G5 of AISI S100 [CSA 136].

B2 Floor and Ceiling Framing

The requirements in Section B2 shall be used in conjunction with the requirements in Section B1, as applicable.

B2.1 Scope

Sections B2.2 through B2.7 are applicable to floor and ceiling systems that utilize *cold-formed steel structural members*.

B2.2 Floor Joist Design

Floor joists shall be designed in accordance with the requirements of this section.

B2.2.1 Bending

Floor joists shall be designed for bending either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. For discretely braced design, *available flexural strength* [*factored resistance*] shall be determined in accordance with Chapter F of AISI S100 [CSA S136].

(b) Continuously Braced Design. For continuously braced design, where *structural sheathing* or *steel deck* is attached to the compression *flange* of the *floor joist* in accordance with Section B2.6(b-2) and the tension *flange* is braced in accordance with Section B2.6(b-3), *available flexural strength* [*factored resistance*] shall be the lesser of the strength determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136]. In considering the distortional buckling strength, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the *floor joist* shall be determined in accordance with Appendix 1.

Floor joists used in an assembly meeting the following conditions are permitted to increase their *available flexural strength* [*factored resistance*] by 15%:

(i) The floor assembly includes identically specified *floor joists* over not less than a 12 ft (3.7 m) width perpendicular to the *floor joists*, and the *floor joists* are at a maximum spacing of 24 in. (610 mm).

(ii) The compression *flange* of each *floor joist* is attached to sheathing with a minimum stiffness and strength equivalent to 3/8 in. (9.5 mm) wood *structural sheathing* that complies with DOC PS 1, DOC PS 2, CSA O437 or CSA O325, or *steel deck* with a minimum profile depth of 9/16 in. (14.3 mm) and a minimum thickness of 0.0269 in. (0.683 mm). The *structural sheathing* or *steel deck* is attached with fasteners having a minimum stiffness and strength equivalent to No. 8 screws, and with fasteners spaced at a maximum of 12 in. (305 mm) on center. The *structural sheathing* or *steel deck* spans at least 3 *floor joists*. The *structural sheathing* or *steel deck* is identified as a structural element in the *construction documents.*

(iii) The tension *flange* of each *floor joist* is braced in accordance with Section B2.6(b-3) of this

Standard or braced in accordance with Section C2.2 of AISI S100 [CSA S136]. At a minimum, continuous *straps* attached to the tension *flange* of each *floor joist* are spaced at a maximum of 12 ft (3.66 m) on center and are at least 1-1/2 inches (38 mm) in width and 33 mils (0.84 mm) in thickness. At a minimum, *straps* are fastened to the bottom *flange* of each *floor joist* with one No. 8 screw and fastened at each end of each *strap* with two No. 8 screws.

(iv) Solid blocking is installed between *floor joists* at each end of each continuous *strap* and at a maximum spacing of 12 ft (3.66 m) perpendicular to the *floor joists*. *Straps* shall be fastened to the solid blocking with a minimum of two No. 8 screws. As an alternative to solid blocking at the ends, the *strap* is permitted to be anchored to a building component with equivalent or greater stiffness to the solid blocking.

B2.2.2 Shear

Shear shall be evaluated in accordance with Sections G2 and G3 (as applicable) of AISI S100 [CSA S136].

B2.2.3 Web Crippling

Web crippling shall be evaluated in accordance with Section B1.6.

B2.2.4 Bending and Shear

The combination of flexure and shear shall be evaluated in accordance with Section H2 of AISI S100 [CSA S136].

B2.2.5 Bending and Web Crippling

The combination of flexure and web crippling shall be evaluated in accordance with Section H3 of AISI S100 [CSA S136], unless a *bearing stiffener* is used in accordance with the requirements of Section B2.5.1.

B2.3 Ceiling Joist Design

Ceiling joists shall be designed in accordance with the requirements of this section.

B2.3.1 Tension

Tension shall be evaluated in accordance with Chapter D of AISI S100 [CSA S136].

B2.3.2 Compression

Ceiling joists shall be designed for compression either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. For discretely braced design, the *available* compression *strength* [*factored resistance*] shall be determined in accordance with Chapter E of AISI S100 [CSA S136]. If the discrete *bracing* restricts rotation of the compression *flanges* about the *web/flange* juncture, the distance between braces shall be used as L_m in determining the distortional buckling force in accordance with Sections 2.3.1.3 and 2.3.2.3 (as applicable) of AISI S100 [CSA S136].

(b) Continuously Braced Design. For continuously braced design, the *available* compression *strength* [*factored resistance*] shall be the lesser of the strength determined in accordance with Section E3 with $F_n = F_y$ or $P_{ne} = P_y$ and the strength determined in accordance with Section E4 of AISI S100 [CSA S136]. In determining the distortional buckling strength, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the *ceiling joist* shall be determined in accordance with Appendix 1.

B2.3.3 Bending

Ceiling joists shall be designed for bending either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. For discretely braced design, *available* flexural *strength* [*factored resistance*] shall be evaluated in accordance with Chapter F of AISI S100 [CSA S136].

(b) Continuously Braced Design. For continuously braced design, where *structural sheathing* or *steel deck* is attached to the compression *flange* of the *ceiling joist* in accordance with Section B2.6(b-2) and the tension *flange* is braced in accordance with Section B2.6(b-3), the *available* flexural *strength* [*factored resistance*] for gravity loading shall be the lesser of the strength determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136]. In determining the distortional buckling moment, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the *ceiling joist* shall be determined in accordance with Appendix 1.

The *available* flexural *strength* [*factored resistance*] for uplift loading shall be evaluated in accordance with Chapter F or Section I6.2.1 (if applicable) of AISI S100 [CSA S136].

B2.3.4 Shear

Shear shall be evaluated in accordance with Sections G2 and G3 (as applicable) of AISI S100 [CSA S136].

B2.3.5 Web Crippling

Web crippling shall be evaluated in accordance with Section B1.6.

B2.3.6 Axial Load and Bending

The combination of axial *load* and bending shall be evaluated in accordance with Section H1 of AISI S100 [CSA S136].

B2.3.7 Bending and Shear

The combination of flexure and shear shall be evaluated in accordance with Section H2 of AISI S100 [CSA S136].

B2.3.8 Bending and Web Crippling

The combination of flexure and web crippling shall be evaluated in accordance with Section H3 of AISI S100 [CSA S136], unless a *bearing stiffener* is used in accordance with the requirements of Section B2.5.1.

B2.4 Floor Truss Design

Floor *trusses* shall be designed in accordance with Chapter E.

B2.5 Bearing Stiffeners

Bearing stiffeners, other than clip angle *bearing stiffeners*, shall be designed in accordance with Section F5 of AISI S100 [CSA S136]. Clip angle *bearing stiffeners*, as permitted in Section B1.6, shall be designed in accordance with Section B2.5.1.

B2.5.1 Clip Angle Bearing Stiffeners

The *nominal* web crippling *strength* [*resistance*] of a *C-shape floor joist* connected to a *rim track* using a clip angle *bearing stiffener* shall be determined in accordance with the following:

$$P_n = 0.9\,(P_j + P_t + 0.5A_gF_y) \qquad (Eq.\ B2.5.1\text{-}1)$$

where

P_j = *Nominal* end two-*flange web* crippling *strength* [*resistance*] of the *floor joist* determined in accordance with Section G5 of AISI S100 [CSA S136]

P_t = *Nominal* interior two-*flange web* crippling *strength* [*resistance*] of the *rim track* determined in accordance with Section G5 of AISI S100 [CSA S136]

A_g = Gross area of the clip angle *bearing stiffener*

F_y = *Yield strength* of clip angle

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω_c = 1.80 for *ASD*

ϕ_c = 0.85 for *LRFD*

ϕ_c = 0.70 for *LSD*

Equation B2.5.1-1 shall be valid for the range of parameters listed in Table B2.5.1-1, as illustrated in Figure B2.5-1.

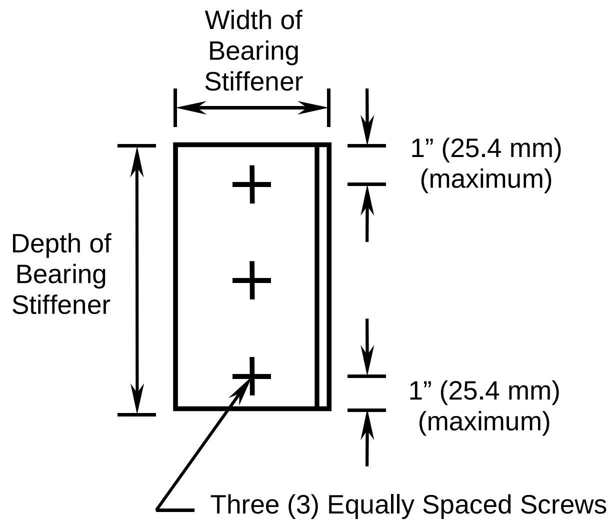

Figure B2.5-1 Fastening of Clip Angle Bearing Stiffener

Table B2.5.1-1
Parameters for Equation B2.5.1-1

Parameter	Minimum	Maximum
Floor Joist and Rim Track:		
Design Thickness	0.0451” (1.146 mm)	0.1017” (2.583 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth of Joist	8” (203 mm)	12” (305 mm)
Bearing Width	1½” (38.1 mm)	n/a
Clip Angle Bearing Stiffener:		
Size	1½”x 1½” (38.1 mm x 38.1 mm)	1½”x 1½” (38.1 mm x 38.1 mm)
Design Thickness	0.0312” (0.792 mm)	0.0713” (1.811 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Length	Joist depth minus 3/8” (9.5 mm)	n/a
Installation:		
Screw Size	No. 8 for angle design thickness less than or equal to 0.0566” (1.438 mm), and No. 10 for thicker angles	n/a
Fasteners	3 screws connecting legs of bearing stiffener to joist and rim track web, in accordance with Figure B2.5-1	n/a

B2.6 Bracing Design

Bracing members shall be designed either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. For discretely braced design, *bracing* members shall be designed in accordance with Section C2.2 of AISI S100 [CSA S136].

(b) Continuously Braced Design. For continuously braced design, *bracing* members shall be designed in accordance with Section C2.2 of AISI S100 [CSA S136], unless the following requirements, as applicable, are met:

  (1) Members are spaced no greater than 24 inches (610 mm) on center.

  (2) The sheathing or deck shall consist of a minimum of 3/8 inch (9.5 mm) wood *structural sheathing* that complies with DOC PS 1, DOC PS 2, CSA O437 or CSA O325, or *steel deck* with a minimum profile depth of 9/16 in. (14.3 mm) and a minimum thickness of 0.0269 in. (0.683 mm). The sheathing or deck shall be attached with minimum No. 8 screws at a maximum 12 inches (305 mm) on center.

  (3) *Floor joists* and *ceiling joists* with simple or continuous spans that exceed 8 feet (2.44 m) shall have the tension *flanges* laterally braced. Each intermediate brace shall be spaced at 8 feet (2.44 m) maximum and shall be designed to resist a required lateral force, P_L, determined in accordance with the following:

  For uniform *loads*:

$$P_L = 1.5(m/d)\,F \qquad (Eq.\ B2.6\text{-}1)$$

where
m = Distance from shear center to mid-plane of *web*
d = Depth of *C-shape* section
F = *w*a
w = Uniform *design load* [*factored load*]
a = Distance between center line of braces

For concentrated *loads*:

If $x \leq 0.3a$:

$$P_L = 1.0(m/d)\ F \qquad (Eq.\ B2.6\text{-}2)$$

If $0.3a < x < 1.0a$:

$$P_L = 1.4(m/d)(1-x/a)\ F \qquad (Eq.\ B2.6\text{-}3)$$

where
x = Distance from concentrated *load* to brace
a = Distance between center line of braces
m = Distance from shear center to mid-plane of *web*
d = Depth of *C-shape* section
F = Concentrated design *load* [*factored load*]

B2.7 Floor Diaphragm Design

Diaphragms shall be designed in accordance with Section B5.

B3 Wall Framing

The requirements in Section B3 shall be used in conjunction with the requirements in Section B1, as applicable.

B3.1 Scope

Sections B3.2 through B3.5 are applicable to wall systems that utilize *cold-formed steel structural members*.

B3.2 Wall Stud Design

Wall *studs* shall be designed in accordance with the requirements of this section.

B3.2.1 Compression

Wall *studs* shall be designed for compression either on the basis of all steel design or on the basis of sheathing braced design with both ends of the *stud* connected to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the *stud* axis, in accordance with the following:

(a) All Steel Design. For all steel design of wall *studs* in compression, Chapter E of AISI S100 [CSA S136] shall define the *available* axial *strength* [*factored resistance*]. The effective length, KL, shall be determined by *rational engineering analysis* or testing, or in the absence of such analysis or tests, K_x, K_y and K_t shall be taken as unity. The unbraced length with respect to the major axis, L_x, shall be taken as the distance between end supports of the member, while unbraced lengths L_y and L_t shall be taken as the distance between braces. If the discrete *bracing* restricts rotation of both *flanges* about the *web*/*flange* juncture, the distance between braces shall be used as L_m in determining the distortional buckling force in accordance with Sections 2.3.1.3 and 2.3.2.3 (as applicable) of AISI S100 [CSA S136] Appendix 2.

(b) Sheathing Braced Design. For sheathing braced design of wall *studs* in compression, the *available* axial *strength* [*factored resistance*] shall be determined in accordance with Section Chapter E of AISI S100 [CSA S136]. The unbraced length with respect to the major axis, L_x, shall be taken as the distance between end supports of the member. The unbraced length with respect to the minor axis, L_y, and the unbraced length for torsion, L_t, shall be taken as twice the distance between sheathing connectors. The buckling coefficients K_x, K_y, and K_t shall be taken as unity. In determining the distortional buckling strength, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the *wall stud* shall be determined in accordance with Appendix 1.

To prevent failure of the sheathing-to-wall *stud connection*, where identical gypsum sheathing is attached to both sides of the wall *stud* with screws spaced at a maximum of 12 inches (305 mm) on center, the maximum axial *nominal load* [*specified load*] in the wall *stud* shall be limited to the values given in Table B3.2-1.

Table B3.2-1
Maximum Axial Nominal Load [Specified Load]
Limited by Gypsum Sheathing-to-Wall Stud Connection Capacity

Gypsum Sheathing	Screw Size	Maximum Nominal [Specified] Stud Axial Load
1/2 inch (12.7 mm)	No. 6	5.8 kips (25.8 kN)
1/2 inch (12.7 mm)	No. 8	6.7 kips (29.8 kN)
5/8 inch (15.9 mm)	No. 6	6.8 kips (30.2 kN)
5/8 inch (15.9 mm)	No. 8	7.8 kips (34.7 kN)

B3.2.2 Bending

Wall *studs* shall be designed for bending either on the basis of all steel design or on the basis of sheathing braced design, in accordance with the following:

(a) All Steel Design. For all steel design, Chapter F of AISI S100 [CSA S136] shall define the *available* flexural *strength* [*factored resistance*].

(b) Sheathing Braced Design. For sheathing braced design, and assuming full rotational restraint provided by the *structural sheathing*, the *available* flexural *strength* [*factored resistance*] shall be the lesser of the strength determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136]. In determining the distortional buckling strength, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the compression *flange* of the *wall stud* shall be determined in accordance with Appendix 1.

B3.2.3 Shear

The *available* shear *strength* [*factored resistance*] shall be determined in accordance with Sections G2 and G3 (as applicable) of AISI S100 [CSA S136].

B3.2.4 Axial Load and Bending

The combination of axial load and bending shall be evaluated in accordance with

Section H1 of AISI S100 [CSA S136].

User Note:
Where the Direct Analysis Method using Amplified First Order Elastic Analysis (Section C1.2 of AISI S100 [CSA S136]) or the Effective Length Method (Section C1.3 of AISI S100 [CSA S136]) is used for members that do not contribute to the stability of the structural system and are considered pinned at both ends, $B_2 \overline{P}_{\ell t}$ and $B_2 \overline{M}_{\ell t}$ can be taken as zero. For this condition, the *required strengths* [*load effects*] are simplified to:

$\overline{M} = B_1 M_{nt}$

$\overline{P} = P_{nt}$

Where notional loads are required per Section C1.3.1.2 of AISI S100 [CSA S136], nominally horizontal or vertical members that do not contribute to the lateral stability of the building and are considered pinned at their ends will not experience *load effects* due to notional loads. Therefore, notional loads need not be considered in the design of these members.

Typical *load*-bearing wall *studs* that do not contribute to the lateral stability of the building are an example of members for which these simplifications would apply.

Variables in this User Note are defined in AISI S100[CSA S136].

B3.2.5 Web Crippling

The *available* web crippling *strength* [*factored resistance*] shall be determined in accordance with Section B1.6, or shall be determined in accordance with Section B3.2.5.1 for *C-shape stud*-to-*track connections* and Section B3.2.5.2 for *C-shape stud*-to-deflection *track connections*.

B3.2.5.1 Stud-to-Track Connection for C-Shape Studs

The *available* web crippling *strength* [*factored resistance*] of *C-shape stud*-to-*track connections* are permitted to be determined in accordance with Items (a) to (c), as applicable:

(a) For single *curtain wall studs* where both *stud flanges* are connected to the *track flanges*, the *nominal strength* [*resistance*], P_{nst}, shall be determined in accordance with Eq. B3.2.5.1-1. For members consisting of two *studs*, P_{nst} is the sum of the individual *stud web* capacities with the *stud design thickness*, t, taken as the thickness of one *web*.

$$P_{nst} = Ct^2F_y\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ B3.2.5.1\text{-}1)$$

where
C = *Web* crippling coefficient listed in Table B3.2.5.1-1
C_R = Inside bend radius coefficient
= 0.19
C_N = Bearing length coefficient
= 0.74
C_h = *Web* slenderness coefficient
= 0.019
R = *Stud* inside bend radius
N = *Stud* bearing length
= *Track flange* width up to a maximum of 2.375 inch (60.3 mm) for single

stud interior configuration

= 1.25 inch (32 mm) for other configurations

h = Depth of flat portion of *stud web* measured along plane of *web*

t = *Stud design thickness*

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 1.70 for *ASD* for single *stud* interior configuration

= 1.90 for *ASD* for all other configurations listed in Table B3.2.5.1-1

ϕ = 0.90 for *LRFD* for single *stud* interior configuration

= 0.85 for *LRFD* for all other configurations listed in Table B3.2.5.1-1

= 0.75 for *LSD* for single *stud* interior configuration

= 0.70 for *LSD* for all other configurations listed in Table B3.2.5.1-1

Equation B3.2.5.1-1 shall be valid for the range of parameters listed in Table B3.2.5.1-2.

Table B3.2.5.1-1
Web Crippling Coefficients

Configuration	Location	Web Crippling Coefficient, C
Single stud	Interior	3.70
Single stud with reinforcing lips facing opening	At a wall opening	2.78
Single stud with stud web facing opening	At a wall opening	1.85
Toe-to-toe double studs	Interior	7.40
Toe-to-toe double studs	At an opening	5.55
Back-to-back double studs	Interior	7.40
Back-to-back double studs	At an opening	7.40

Table B3.2.5.1-2
Parameters for Equation B3.2.5.1-1

Parameter	Minimum	Maximum
Screw Size	No. 8 for 0.0451 in. (1.146 mm) and thinner stud design thickness No. 10 for 0.0452 to 0.0566 in. (1.147 to 1.438 mm) stud design thickness No. 12 for 0.0567 in. (1.439 mm) and thicker stud design thickness	n/a
Stud Section:		
Design Thickness	0.0346 inch (0.88 mm)	0.0770 inch (1.96 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Track Section:		
Design Thickness	0.0346 inch (0.88 mm)	0.0770 inch (1.96 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Nominal Flange Width	1.25 inch (31.8 mm)	2.375 inch (60.3 mm)

(b) For single *curtain wall studs* that are not adjacent to wall openings and where both *stud flanges* are connected to the *track flanges* and the *track* thickness is less than the *stud* thickness, the *nominal strength* [*resistance*], P_{nst}, shall be the lesser value determined in accordance with Equation B3.2.5.1-1 or B3.2.5.1-2.

$$P_{nst} = 0.6\, t_t\, w_{st}\, F_{ut} \qquad (Eq.\ B3.2.5.1\text{-}2)$$

where:

t_t = *Track design thickness*

w_{st} = $20\, t_t + 0.56\alpha$

α = Coefficient for conversion of units
= 1.0 when t_t is in inches
= 25.4 when t_t is in mm

F_{ut} = Tensile strength of the *track*

P_{nst} = *Nominal strength* [*resistance*] for the *stud*-to-*track connection* when subjected to transverse *loads*

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 1.70 for *ASD*

ϕ = 0.90 for *LRFD*
= 0.80 for *LSD*

Equation B3.2.5.1-2 shall be valid for the range of parameters listed in Table B3.2.5.1-3.

Table B3.2.5.1-3
Parameters for Equation B3.2.5.1-2

Parameter	Minimum	Maximum
Screw Size	No. 8	n/a
Stud Section:		
Design Thickness	0.0346 inch (0.88 mm)	0.0770 inch (1.96 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Track Section:		
Design Thickness	0.0346 inch (0.88 mm)	0.0770 inch (1.96 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Nominal Flange Width	1.25 inch (31.8 mm)	2.375 inch (60.3 mm)

(c) For *curtain wall* jamb *studs* made up of two *studs* connected back-to-back where both *stud flanges* are connected to the *track flanges* and the *track* thickness is the same as the *stud* thickness, the *nominal strength* [*resistance*], P_{nst}, is the lesser value determined in accordance with Equation B3.2.5.1-1 or B3.2.5.1-3.

$$P_{nst} = 15.2\, t_t^2\, F_{ut} \qquad (Eq.\ B3.2.5.1\text{-}3)$$

where

t_t = *Track design thickness*

F_{ut} = Tensile strength of the *track*

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 2.10 for *ASD*

ϕ = 0.75 for *LRFD*

= 0.65 for *LSD*

Equation B3.2.5.1-3 shall be valid for the range of parameters listed in Table B3.2.5.1-4.

Table B3.2.5.1-4
Parameters for Equation B3.2.5.1-3

Parameter	Minimum	Maximum
Screw Size	No. 10	n/a
Stud Section:		
Design Thickness	0.0346 inch (0.88 mm)	0.0770 inch (1.96 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Track Section:		
Design Thickness	0.0346 inch (0.88 mm)	0.0770 inch (1.96 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Nominal Flange Width	1.25 inch (31.8 mm)	1.25 inch (31.8 mm)

(d) For *curtain wall studs* that are not adjacent to wall openings and do not have both *stud flanges* connected to the *track flanges* and the *track* thickness is greater than or equal to the *stud* thickness, *nominal strength* [*resistance*], P_{nst} shall equal P_n, along with Ω and ϕ, as determined by Section G5 of AISI S100 [CSA S136]. For two *studs* connected back-to-back, the two members shall be considered individually as single *web* members.

(e) For *curtain wall studs* that are adjacent to wall openings and do not have both *stud flanges* connected to the *track flanges* and the *track* thickness is greater than or equal to the *stud* thickness, *nominal strength* [*resistance*], P_{nst} shall equal $0.5P_n$, along with Ω and ϕ, as determined by Section G5 of AISI S100 [CSA S136]. For two *studs* connected back-to-back, the two members shall be considered individually as single *web* members.

B3.2.5.2 Deflection Track Connection for C-Shape Studs

The *available strength* [*factored resistance*] of *deflection track connections* for *C-shape studs* shall be determined in accordance with Sections B3.2.5.2.1 and B3.2.5.2.2, as applicable.

B3.2.5.2.1 For *curtain wall studs* used in *deflection track connections,* the *nominal* web crippling *strength* [*resistance*], P_{nst} and the Ω and ϕ factors shall be determined by Section G5 of AISI S100 [CSA S136]. The bearing length used in these calculations shall not exceed the minimum engagement between the *stud* and the *track* or 1 inch (25.4 mm).

B3.2.5.2.2 The *nominal strength* [*resistance*] of a single *deflection track* subjected to transverse *loads* and connected to its support at a fastener spacing not greater than the *stud* spacing shall be determined in accordance with Eq. B3.2.5.2-1.

$$P_{ndt} = \frac{w_{dt} t_t^2 F_y}{4e} \qquad (Eq.\ B3.2.5.2\text{-}1)$$

where

w_{dt} = Effective *track* length

$= 0.11(\alpha^2)(e^{0.5}/t_t^{1.5}) + 5.5\alpha \leq S \qquad (Eq.\ B3.2.5.2\text{-}2)$

S = Center-to-center spacing of *studs*

t_t = *Track design thickness*

F_y = Design *yield strength* of *track* material

e = Design end or slip gap (distance between *stud web* at end of *stud* and *track web*)

α = Coefficient for conversion of units

= 1.0 when e, t_t and S are in inches

= 25.4 when e, t_t and S are in mm

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 2.80 for *ASD*

ϕ = 0.55 for *LRFD*

= 0.45 for *LSD*

Equation B3.2.5.2-1 shall be valid for the range of parameters listed in Table B3.2.5.2-1. Additionally, the horizontal distance from the *web* side of the *stud* to the terminating end of the *track* shall not be less than one-half the effective *track* length w_{dt}.

Table B3.2.5.2-1
Parameters for Equation B3.2.5.2-1

Parameter	Minimum	Maximum
Screw Size	No. 10	n/a
Stud Section:		
Design Thickness	0.0451 inch (1.14 mm)	0.0713 inch (1.81 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Nominal Flange Width	1.625 inch (41.3 mm)	2.50 inch (63.5 mm)
Stud Spacing	12 inch (305 mm)	24 inch (610 mm)
Stud Bearing Length	¾ inch (19.1 mm)	n/a
Track Section:		
Design Thickness	0.0451 inch (1.14 mm)	0.0713 inch (1.81 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
Nominal Depth	3.50 inch (88.9 mm)	6.0 inch (152.4 mm)
Nominal Flange Width	2.00 inch (50.8 mm)	3.00 inch (76.3 mm)

B3.3 Header Design

B3.3.1 Back-to-Back Headers

The requirements of this section shall be limited to back-to-back *header* beams that are installed using *cold-formed steel C-shape* sections in accordance with Section C3.4.4.

B3.3.1.1 Bending

Bending shall be evaluated in accordance with Section F3 of AISI S100 [CSA S136] with $F_n = F_y$ or $M_{ne} = M_y$.

B3.3.1.2 Shear

Shear shall be evaluated in accordance with Sections G2 and G3 (as applicable) of AISI S100 [CSA S136].

B3.3.1.3 Web Crippling

Web crippling shall be evaluated in accordance with Sections G5 and G6 (as applicable) of AISI S100 [CSA S136]. For back-to-back *header* beams, the coefficients for built-up sections in Table G5-1 of AISI S100 [CSA S136] shall be used.

B3.3.1.4 Bending and Shear

The combination of bending and shear shall be evaluated in accordance with Section H2 of AISI S100 [CSA S136].

B3.3.1.5 Bending and Web Crippling

Webs of back-to-back *header* beams subjected to a combination of bending and *web* crippling shall be designed in accordance with Section H3 of AISI S100 [CSA S136].

B3.3.2 Box Headers

The requirements of this section shall be limited to box *header* beams that are installed using *cold-formed steel C-shape* sections in accordance with Section C3.4.4.

B3.3.2.1 Bending

Bending shall be evaluated in accordance with Section F3 of AISI S100 [CSA S136] with $F_n = F_y$ or $M_{ne} = M_y$.

B3.3.2.2 Shear

Shear shall be evaluated in accordance with Sections G2 and G3 (as applicable) of AISI S100 [CSA S136].

B3.3.2.3 Web Crippling

Web crippling shall be evaluated in accordance with G5 and G6 (as applicable) of AISI S100 [CSA S136]. For box *header* beams, the equations for shapes having single *webs* shall be used. The *nominal web* crippling *strength* [*resistance*], P_n, for an interior one-*flange* loading condition, with the applicable Ω or ϕ factor specified below, is permitted to be multiplied by α, where α accounts for the increased strength due to the *track* and is specified as follows:

α = Parameter determined in accordance with Equation B3.3.2.3-1 or B3.3.2.3-2

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 1.80 for *ASD*

ϕ = 0.85 for *LRFD*

ϕ = 0.80 for *LSD*

Where the *track* section *design thickness* ≥ 0.0346 in. (0.879 mm), the *track flange* width ≥ 1 in. (25.4 mm), the *C-shape* section depth ≤ 12 in. (305 mm) and the *C-shape* section *design thickness* ≥ 0.0346 in. (0.879 mm):

$$\alpha = 2.3\frac{t_t}{t_c} \geq 1.0 \qquad (Eq.\ \text{B3.3.2.3-1})$$

where

t_t = 0.0346 in. (0.879 mm)

t_c = *Design thickness* of the *C-shape* section

Where the limits of Equation B3.3.2.3-1 are not met:

$$\alpha = 1.0 \qquad (Eq.\ \text{B3.3.2.3-2})$$

B3.3.2.4 Bending and Shear

The combination of bending and shear shall be evaluated in accordance with Section H2 of AISI S100 [CSA S136].

B3.3.2.5 Bending and Web Crippling

Webs of box *header* beams subjected to a combination of bending and *web* crippling shall be determined in accordance with either Section H3 of AISI S100 [CSA S136] or the following equations:

(a) For *ASD*:

$$\frac{P}{P_n} + \frac{M}{M_n} \le \frac{1.5}{\Omega} \qquad (Eq.\ B3.3.2.5\text{-}1)$$

where

P = *Required* web crippling *strength*

M = *Required* flexural *strength*

P_n = *Nominal web* crippling *strength* determined in accordance with Section B3.3.2.3

M_n = *Nominal flexural strength* defined in Section F3 of AISI S100 [CSA S136] with $F_n = F_y$ or $M_{ne} = M_y$

The *available strength* shall be determined using the *safety factor* (Ω) as follows:

Ω = 1.85

(b) For *LRFD* and *LSD*:

$$\frac{P_u}{P_n} + \frac{M_u}{M_n} \le 1.5\phi \qquad (Eq.\ B3.3.2.5\text{-}2)$$

where

P_u = *Required web* crippling *strength* [compression force due to *factored loads*]

M_u = *Required* flexural *strength* [moment due to *factored loads*]

P_n = *Nominal web* crippling *strength* [*resistance*] computed by Section B3.3.2.3

M_n = *Nominal* flexural *strength* [resistance] defined in Section F3 of AISI S100 [CSA S136] with $F_n = F_y$ or $M_{ne} = M_y$

The *available strength* [*factored resistance*] shall be determined using the *resistance factor* (ϕ) as follows:

ϕ = 0.85 for *LRFD*
= 0.80 for *LSD*

B3.3.3 Double L-Headers

The requirements of this section shall be limited to double L-*headers* that are installed using two *cold-formed steel* angles in accordance with Section C3.4.5 and meet all of the following conditions:

(a) Minimum top *flange* width = 1.5 inches (38.1 mm)
(b) Maximum vertical leg dimension = 10 inches (254 mm)
(c) Minimum *base steel thickness* = 0.033 inches (0.838 mm)
(d) Maximum *design thickness* = 0.0713 inches (1.829 mm)
(e) Minimum design *yield strength*, F_y = 33 ksi (230 MPa)
(f) Maximum design *yield strength*, F_y = 50 ksi (345 MPa)
(g) Cripple *stud* located at all *load* points
(h) Minimum bearing length 1.5 inches (38.1 mm) at *load* points
(i) Minimum wall width = 3.5 inches (88.9 mm)
(j) Maximum *span* = 16 ft-0 in. (4.88 m)

B3.3.3.1 Bending

The *available* flexural *strength* [*factored resistance*] of double L-*headers* shall be determined in accordance with this section.

B3.3.3.1.1 Gravity Loading

The gravity *nominal* flexural *strength* [*resistance*], M_{ng}, shall be determined as follows:

$M_{ng} = M_{n\ell o}$ when $L/L_h \geq 10$ (*Eq*. B3.3.3.1.1-1)

$= 0.9\, M_{n\ell o}$ when $L/L_h < 10$ and $L_h > 8$ in. (203 mm) (*Eq*. B3.3.3.1.1-2)

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

For $L_h \leq 8$ in. (203 mm):

Ω = 1.67 (*ASD*)
ϕ = 0.90 (*LRFD*)
= 0.85 (*LSD*)

For $L_h > 8$ in. (203 mm):

Ω = 2.25 (*ASD*)
ϕ = 0.70 (*LRFD*)
= 0.65 (*LSD*)

where

$M_{n\ell o}$ = *Nominal strength* [*resistance*] determined in accordance with Section F3 of AISI S100 [CSA, S136] with $F_n = F_y$ or $M_{ne} = M_y$
L = Span length
L_h = Vertical leg dimension of angle

B3.3.3.1.2 Uplift Loading

The *nominal* uplift flexural *strength* [*resistance*], M_{nu}, shall be determined as follows:

$M_{nu} = R\, M_{ng}$ (*Eq*. B3.3.3.1.2-1)

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 2.0 (*ASD*)
ϕ = 0.80 (*LRFD*)
= 0.75 (*LSD*)

where

M_{ng} = Gravity *nominal* flexural *strength* [*resistance*] determined by Eq. B3.3.3.1.1-1 or B3.3.3.1.1-2
R = Uplift factor
= 0.25 for $L_h/t \leq 150$
= 0.20 for $L_h/t \geq 170$
= $0.25 - 0.0025(L_h/t - 150)$ for $150 < L_h/t < 170$ (*Eq*. B3.3.3.1.2-2)
L_h = Vertical leg dimension of the angle
t = *Design thickness* of L-*header* angle

B3.3.3.2 Shear

Shear need not be evaluated for the design of double L-*header* beams that are fabricated and installed in accordance with this Standard.

B3.3.3.3 Web Crippling

Web crippling need not be evaluated for the design of double L-*header* beams that are fabricated and installed in accordance with this Standard.

B3.3.3.4 Bending and Shear

The combination of bending and shear need not be evaluated for the design of double L-*header* beams fabricated and installed in accordance with this Standard.

B3.3.3.5 Bending and Web Crippling

The combination of bending and *web* crippling need not be evaluated for the design of double L-*header* beams fabricated and installed in accordance with this Standard.

B3.3.4 Single L-Headers

The requirements of this section shall be limited to single L-*headers* that are installed using one *cold-formed steel* angle in accordance with Section C3.4.5 and meet all of the following conditions:

(a) Minimum top *flange* width = 1.5 inches (38.1 mm)
(b) Maximum vertical leg dimension = 8 inches (203 mm)
(c) Minimum *base steel thickness* = 0.033 inches (0.838 mm)
(d) Maximum *design thickness* = 0.0566 inches (1.448 mm)
(e) Minimum design *yield strength*, F_y = 33 ksi (230 MPa)
(f) Maximum design *yield strength*, F_y = 50 ksi (345 MPa)
(g) Cripple *studs* located at all *load* points
(h) Minimum bearing length 1.5 inches (38.1 mm) at *load* points
(i) Minimum wall width = 3.5 inches (88.9 mm)
(j) Maximum *span* = 4′-0″ (1.219 m)

B3.3.4.1 Bending

The *available* flexural *strength* [*factored resistance*] of single L-*headers* shall be determined in accordance with this section.

B3.3.4.1.1 Gravity Loading

The gravity *nominal* flexural *strength* [*resistance*], M_{ng}, shall be determined as follows:

$$M_{ng} = M_{n\ell o} \quad \text{when } L_h \leq 6 \text{ in. (152 mm)} \qquad (Eq. \text{ B3.3.4.1.1-1})$$

$$= 0.9\, M_{n\ell o} \quad \text{when 6 in. (152 mm)} < L_h \leq 8 \text{ in. (203 mm)} \qquad (Eq. \text{ B3.3.4.1.1-2})$$

The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as follows:

Ω = 1.67 (*ASD*)
ϕ = 0.90 (*LRFD*)
= 0.85 (*LSD*)

where

$M_{n\ell o}$ = *Nominal strength* [*resistance*] determined in accordance with Section F3 of AISI S100 [CSA, S136] with $F_n = F_y$ or $M_{ne} = M_y$

L_h = Vertical leg dimension of angle

B3.3.4.1.2 Uplift Loading

[Reserved]

B3.3.4.2 Shear

Shear need not be evaluated for the design of single L-*header* beams that are fabricated and installed in accordance with this Standard.

B3.3.4.3 Web Crippling

Web crippling need not be evaluated for the design of single L-*header* beams that are fabricated and installed in accordance with this Standard.

B3.3.4.4 Bending and Shear

The combination of bending and shear need not be evaluated for the design of single L-*header* beams fabricated and installed in accordance with this Standard.

B3.3.4.5 Bending and Web Crippling

The combination of bending and *web* crippling need not be evaluated for the design of single L-*header* beams fabricated and installed in accordance with this Standard.

B3.3.5 Inverted L-Header Assemblies

B3.3.5.1 The requirements of this section shall be limited to inverted double or single L-*header* assemblies satisfying the limitations specified in Section B3.3.3 for double L-*headers* and Section B3.3.4 for single L-*headers*, respectively. The installation of inverted L-*header* assemblies shall be in accordance with Section C3.4.6.

B3.3.5.2 For double inverted L-*header* assemblies, the *nominal* flexural *strength* [*resistance*] of the combined L-*header* assembly (i.e., double L-*headers* plus inverted double L-*headers*) shall be determined by summing the gravity and uplift *nominal* flexural *strengths* [*resistances*] as determined in accordance with Section B3.3.3.1.

B3.3.5.3 For single inverted L-*header* assemblies, the *nominal* flexural *strength* [*resistance*] of the combined L-*header* assembly (i.e., a single L-*header* plus an inverted single L-*header*) shall be based on the gravity *nominal* flexural *strength* [*resistance*] as determined in accordance with Section B3.3.4.1.

B3.3.5.4 Shear, *web* crippling, bending and shear, and bending and *web* crippling need not be evaluated for the design of inverted L-*headers* fabricated and installed in accordance with this Standard.

B3.4 Bracing

B3.4.1 Intermediate Brace Design

B3.4.1.1 For bending members, each intermediate brace shall be designed in accordance with Section C2.2.1 of AISI S100 [CSA S136].

B3.4.1.2 For axial loaded members, each intermediate brace shall be designed for 2% of the design compression force in the member.

B3.4.1.3 For combined bending and axial *loads*, each intermediate brace shall be designed for the combined brace force determined in accordance with Section C2.2.1 of AISI S100 [CSA S136] and 2% of the design compression force in the member.

B3.5 Serviceability

Serviceability limits shall be chosen based on the intended function of the wall system, and shall be evaluated using *load* and *load* combinations in accordance with Section B1.1.

B4 Roof Framing

The requirements in Section B4 shall be used in conjunction with the requirements in Section B1, as applicable.

B4.1 Scope

Sections B4.2 through B4.6 are applicable to roof systems that utilize *cold-formed steel structural members*.

B4.2 Roof Rafter Design

Roof rafters shall be designed in accordance with the requirements of this section.

B4.2.1 Tension

Tension shall be evaluated in accordance with Chapter D of AISI S100 [CSA S136].

B4.2.2 Compression

Roof rafters shall be designed for compression either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. For discretely braced design, the *available* compression *strength* [*factored resistance*] shall be determined in accordance with Chapter E of AISI S100 [CSA S136]. In determining the distortional buckling strength, if the discrete *bracing* restricts rotation of the compression *flange* about the *web/flange* juncture, the distance between braces shall be used as L_m when applying AISI S100 [CSA S136].

(b) Continuously Braced Design. For continuously braced design, the *available* compression *strength* [*factored resistance*] shall be the lesser of the strength determined in accordance with Section E3 with $F_n = F_y$ or $P_{ne} = P_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136]. In determining the distortional buckling strength, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the *roof rafter* shall be determined in accordance with Appendix 1.

B4.2.3 Bending

Roof rafters shall be designed for bending either on the basis of discretely braced design or on the basis of continuously braced design, in accordance with the following:

(a) Discretely Braced Design. For discretely braced design, the *available* flexural *strength* [*factored resistance*] shall be determined in accordance with Chapter F of AISI S100 [CSA S136].

(b) Continuously Braced Design. For continuously braced design, where *structural sheathing* or *steel deck* is attached to the compression *flange* of the *roof rafter* in accordance with Section B4.5(b) and the tension *flange* is braced in accordance with Section B4.5(c), the *available* flexural *strength* [*factored resistance*] for gravity loading shall be determined by using Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and Section F4 of AISI S100 [CSA S136]. The *available* flexural *strength* [*factored resistance*] for uplift loading shall be determined in accordance with Chapter F or Section I6.2.1 (if applicable) of AISI S100 [CSA S136]. Where *structural sheathing* or *steel deck* is attached to both *flanges* of the *roof rafter* in accordance with Section B4.5(b), flexure for gravity or uplift loading shall be determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and Section F4 of AISI S100 [CSA S136]. In determining the distortional buckling strength, the rotational stiffness, k_ϕ, provided by the *structural sheathing* or *steel deck* to the *roof rafter* shall be determined in accordance with Appendix 1.

B4.2.4 Shear

Shear shall be evaluated in accordance with Sections G2 and G3 (as applicable) of AISI S100 [CSA S136].

B4.2.5 Web Crippling

Web crippling shall be evaluated in accordance with Section B1.6.

B4.2.6 Axial Load and Bending

The combination of axial *load* and bending shall be evaluated in accordance with Section H1 of AISI S100 [CSA S136].

B4.2.7 Bending and Shear

The combination of flexure and shear shall be evaluated in accordance with Section H2 of AISI S100 [CSA S136].

B4.2.8 Bending and Web Crippling

The combination of flexure and *web* crippling shall be evaluated in accordance with Section H3 of AISI S100 [CSA S136], unless a *bearing stiffener* is used in accordance with the requirements of Section B4.4.

B4.3 Roof Truss Design

Roof *trusses* shall be designed in accordance with Chapter E.

B4.4 Bearing Stiffeners

Bearing stiffeners shall be designed in accordance with Section F5 of AISI S100 [CSA S136].

B4.5 Bracing Design

Bracing members shall be designed in accordance with Section C2 of AISI S100 [CSA S136], unless the following requirements, as applicable, are met:

(a) Members are spaced no greater than 24 inches (610 mm) on center.

(b) In continuously braced design, the sheathing or deck shall consist of a minimum of 3/8 in. (9.5 mm) wood *structural sheathing* that complies with DOC PS 1, DOC PS 2, CSA O437 or CSA O325, or *steel deck* with a minimum profile depth of 9/16 in. (14.3 mm) and a minimum thickness of 0.0269 in. (0.683 mm). The sheathing or deck shall be attached with minimum No. 8 screws at a maximum 12 inches (305 mm) on center.

(c) In continuously braced design, *roof rafters* with simple or continuous spans that exceed 8 feet (2.44 m) shall have the tension *flanges* laterally braced. Each intermediate brace shall be spaced at 8 feet (2.44 m) maximum and shall be designed to resist a required lateral force, P_L, determined in accordance with the following:

(1) For uniform *loads*:

$$P_L = 1.5(m/d)\ F \qquad (Eq.\ B4.5\text{-}1)$$

where
m = Distance from shear center to mid-plane of *web*
d = Depth of *C-shape* section
F = wa
w = Uniform *design load* [*factored load*]
a = Distance between center line of braces

(2) For concentrated *loads*:

If $x \le 0.3a$

$$P_L = 1.0(m/d)\ F \qquad (Eq.\ B4.5\text{-}2)$$

If $0.3a < x < 1.0a$

$$P_L = 1.4(m/d)(1\text{-}x/a)\ F \qquad (Eq.\ B4.5\text{-}3)$$

where
m = Distance from shear center to mid-plane of *web*
d = Depth of *C-shape* section
F = Concentrated *design load* [*factored load*]
x = Distance from concentrated *load* to brace
a = Distance between center line of braces

B4.6 Roof Diaphragm Design

Diaphragms shall be designed in accordance with Section B5.4.

B5 Lateral Force-Resisting Systems

The requirements in Section B5 shall be used in conjunction with the requirements in Section B1, as applicable.

B5.1 Scope

Sections B5.2 through B5.5 are applicable to buildings that utilize *cold-formed steel structural members* for *lateral force-resisting system* framing.

User Note:
See Section A1.2.2 for applicability of this section to *cold-formed steel structural members* and connections in *seismic force-resisting systems*.

B5.2 Shear Wall Design

Shear walls shall be designed as either *Type I shear walls* or *Type II shear walls* in accordance with the requirements of this section and shall be installed in accordance with the requirements of Section C3.6.1.

User Note:
See Section A1.2.2 for applicability of this section to *cold-formed steel structural members* and

connections in *seismic force-resisting systems*.

B5.2.1 General

Type I shear walls shall be fully sheathed with *steel sheet sheathing*, *wood structural panels*, gypsum board panels, or *fiberboard* panels with *hold-downs* at each end. *Type I shear walls* sheathed with *steel sheet sheathing* or *wood structural panels* are permitted to have openings where details are provided to account for force transfer around openings. *Type I shear walls* shall conform to the additional requirements of Section B5.2.1.1.

Type II shear walls shall be sheathed with *steel sheet sheathing* or *wood structural panels* with a *Type II shear wall segment* at each end. Openings are permitted to occur beyond the ends of the *Type II shear wall*; however, the width of such openings shall not be included in the length of the *Type II shear wall*. *Type II shear walls* shall conform to the additional requirements of Section B5.2.1.2.

B5.2.1.1 Type I Shear Walls

Type I shear walls shall conform to the following requirements:

(a) The height-to-length aspect ratio (h/w) of a *Type I shear wall* does not exceed the values in Tables B5.2.2.3-1 through B5.2.2.3-4.

(b) The length of a *Type I shear wall* is not less than 24 inches (610 mm).

(c) The height-to-length aspect ratio (h_p/w_p) of a *wall pier* in a *Type I shear wall* with openings is limited to a maximum of 2:1, where the height of a *wall pier* (h_p) is defined as the height of the opening adjacent to the sheathed wall and the length of a *wall pier* (w_p) is the sheathed length of the *wall pier* adjacent to the opening.

(d) The length of a *wall pier* (w_p) in a *Type I shear wall* with openings is not less than 24 in. (610 mm).

B5.2.1.2 Type II Shear Walls

Type II shear walls shall conform to the following requirements:

(a) The height-to-length aspect ratio (h/w) of a *Type II shear wall segment* does not exceed the values in Tables B5.2.2.3-1 through B5.2.2.3-2.

(b) For a *Type II shear wall,* the *nominal strength* [*resistance*] for shear, V_n, is based upon a screw spacing of not less than 4 inches (100 mm) on center.

(c) Where horizontal out-of-plane offset irregularities occur in a *Type II shear wall*, portions of the wall on each side of the offset irregularity are designated as separate *Type II shear walls*.

(d) *Collectors* for shear transfer are provided for the full length of the *Type II shear wall*.

(e) *Type II shear walls* have uniform top-of-wall and bottom-of-wall elevations.

(f) *Type II shear wall* height, h, does not exceed 20 feet (6.1 m).

B5.2.2 Nominal Strength [Resistance]

B5.2.2.1 Type I Shear Walls

The *nominal strength* [*resistance*] for shear, V_n, shall be determined in accordance with the following:

For $h/w \le 2$,

$V_n = v_n w$ (*Eq*. B5.2.2-1)

where

h = Height of the *shear wall*, ft (m)

w = Length of the *shear wall*, ft (m)

v_n= *Nominal strength* [*resistance*] per unit length as specified in Section B5.2.2.3, lbs/ft (kN/m)

Where permitted in Tables B5.2.2.3-1, B5.2.2.3-2 and Section B5.2.2.3.2.1, the *nominal strength* [*resistance*] for shear, V_n, for height-to-length aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, shall be determined in accordance with the following:

For $2 < h/w \leq 4$,

$V_n = v_n w(2w/h)$ (*Eq*. B5.2.2-2)

B5.2.2.2 Type II Shear Walls

The *nominal strength* [*resistance*] for shear, V_n, shall be determined in accordance with the following:

$V_n = C_a v_n \Sigma L_i$ (Eq. B5.2.2-3)

where

C_a = Shear resistance adjustment factor from Table B5.2.2.2-1. For intermediate values of opening height ratio and percentages of full-height sheathing, the shear resistance adjustment factors are permitted to be determined by interpolation.

v_n = *Nominal strength* [*resistance*] per unit length as specified in Section B5.2.2.3, lbs/ft (kN/m)

ΣL_i = Sum of lengths of *Type II shear wall segments,* ft (m)

The percent of full-height sheathing shall be calculated as the sum of lengths (ΣL_i) of *Type II shear wall segments* divided by the total length of the *Type II shear wall* including openings.

The maximum opening height ratio shall be calculated by dividing the maximum opening clear height by the *shear wall* height, h.

Table B5.2.2.2-1
Shear Resistance Adjustment Factor, C_a

	Maximum Opening Height Ratio				
	1/3	1/2	2/3	5/6	1
Percent Full-Height Sheathing	**Shear Resistance Adjustment Factor**				
10%	1.00	0.69	0.53	0.43	0.36
20%	1.00	0.71	0.56	0.45	0.38
30%	1.00	0.74	0.59	0.49	0.42
40%	1.00	0.77	0.63	0.53	0.45
50%	1.00	0.80	0.67	0.57	0.50
60%	1.00	0.83	0.71	0.63	0.56
70%	1.00	0.87	0.77	0.69	0.63
80%	1.00	0.91	0.83	0.77	0.71
90%	1.00	0.95	0.91	0.87	0.83
100%	1.00	1.00	1.00	1.00	1.00

B5.2.2.3 Nominal Strength [Resistance] per Unit Length

B5.2.2.3.1 General Requirements

Type I shear walls and *Type II shear walls* shall conform to the following requirements:

(a) Wall *studs* and *track* are ASTM A1003 Structural Grade 33 (Grade 230) Type H steel for members with a *designation thickness* of 33 and 43 mils, and ASTM A1003 Structural Grade 50 (Grade 340) Type H steel for members with a *designation thickness* equal to or greater than 54 mils.

(b) *Studs* are *C-shape* members with a minimum *flange* width of 1-5/8 in. (41.3 mm), minimum *web* depth of 3-1/2 in. (89 mm) and minimum *edge stiffener* of 3/8 in. (9.5 mm).

(c) *Track* has a minimum *flange* width of 1-1/4 in. (31.8 mm) and a minimum *web* depth of 3-1/2 in. (89 mm).

(d) Screws for *structural members* are a minimum No. 8 and are in accordance with ASTM C1513.

(e) Panels less than 12 inches (305 mm) wide are not used.

(f) Maximum *stud* spacing is 24 inches (610 mm) on center.

(g) All sheathing edges are attached to *structural members* or *panel blocking*, unless otherwise specified.

(h) Where used as *panel blocking*, flat *strap* is a minimum thickness of 33 mils with a minimum width of 1-1/2 inches (38.1 mm).

B5.2.2.3.2 Steel Sheet Sheathing

The *nominal strength* [*resistance*] per unit length for assemblies with *steel sheet sheathing* and *panel blocking* shall be as specified in Table B5.2.2.3-1 or determined in accordance with Section B5.2.2.3.2.1.

The *nominal strength* [*resistance*] per unit length for unblocked assemblies with panel edges overlapped and attached to each other with screw spacing as required for panel edges in lieu of *panel blocking* shall be as specified in Table B5.2.2.3-1 multiplied by 0.70.

Tabulated values in Table B5.2.2.3-1 shall be applicable for any *load* duration.

Systems tabulated in Table B5.2.2.3-1 shall conform to the following additional requirements:

(a) *Steel sheet sheathing* has a minimum *designation thickness* as specified in Table B5.2.2.3-1 and complies with ASTM A1003 Structural Grade 33 (Grade 230) Type H.

(b) In Canada, *steel sheet sheathing* is connected without horizontal joints.

(c) Screws used to attach *steel sheets* are a minimum No. 8 and are in accordance with ASTM C1513.

B5.2.2.3.2.1 Effective Strip Method

The effective strip method is permitted to be used only in the United States and Mexico. The *nominal shear strength* [*resistance*] per unit length for assemblies with *steel sheet sheathing* shall be determined in accordance with the following:

$$v_n = \text{minimum}(1.23P_n \cos\alpha / w,\ 1.23w_e tF_y \cos\alpha / w) \qquad (Eq.\ B5.2.2.3.2.1\text{-}1)$$

where

P_n = *Nominal* shear *strength* [*resistance*] of screw *connections* at the sheet edge developing the tension strength at one end of the effective strip width, w_e, on the *steel sheet sheathing*

α = Arctan(h/w) (*Eq.* B5.2.2.3.2.1-2)

t = *Design thickness* of *steel sheet sheathing*

F_y = *Yield strength* of *steel sheet sheathing*

w_e = w_{max} when $\lambda \le 0.0819$ (*Eq.* B5.2.2.3.2.1-3)

= ρw_{max} when $\lambda > 0.0819$ (*Eq.* B5.2.2.3.2.1-4)

where

w_{max} = $w/\sin\alpha$ (*Eq.* B5.2.2.3.2.1-5)

$$\rho = \frac{1 - 0.55(\lambda - 0.08)^{0.12}}{\lambda^{0.12}} \qquad (Eq.\ B5.2.2.3.2.1\text{-}6)$$

$$\lambda = 1.736\frac{\alpha_1 \alpha_2}{\beta_1 \beta_2 \beta_3^2 a} \qquad (Eq.\ B5.2.2.3.2.1\text{-}7)$$

where

α_1 = $F_{ush}/45$ (For F_{ush} in ksi) (*Eq.* B5.2.2.3.2.1-8)

= $F_{ush}/310.3$ (For F_{ush} in MPa) (*Eq.* B5.2.2.3.2.1-9)

α_2 = $F_{uf}/45$ (For F_{uf} in ksi) (*Eq.* B5.2.2.3.2.1-10)

= $F_{uf}/310.3$ (For F_{uf} in MPa) (*Eq.* B5.2.2.3.2.1-11)

β_1 = $t_{sh}/0.018$ (For t_{sh} in inches) (*Eq.* B5.2.2.3.2.1-12)

= $t_{sh}/0.457$ (For t_{sh} in mm) (*Eq.* B5.2.2.3.2.1-13)

β_2 = $t_f/0.018$ (For t_f in inches) (*Eq.* B5.2.2.3.2.1-14)

= $t_f/0.457$ (For t_f in mm) (*Eq.* B5.2.2.3.2.1-15)

β_3 = $s/6$ (For s in inches) (*Eq.* B5.2.2.3.2.1-16)

= $s/152.4$ (For s in mm) (*Eq.* B5.2.2.3.2.1-17)

F_{ush} = *Tensile strength* of *steel sheet sheathing*

F_{uf} = Minimum *tensile strength* of framing materials

t_{sh} = *Design thickness* of *steel sheet sheathing*

t_f = Minimum *design thicknesses* of framing members

s = Screw spacing on the panel edges

a = Wall aspect ratio (h:w)

= h/w (*Eq*. B5.2.2.3.2.1-18)

Other variables are defined in Section B5.2.2.1.

The Effective Strip Method is permitted to be used within the following range of parameters:

(a) *Designation thickness* of *stud*, *track*, and *stud blocking*: a minimum of 33 mils (0.838 mm).

(b) *Designation thickness* of *steel sheet sheathing*: 18 mils (0.457 mm) to 33 mils (0.838 mm).

(c) Screw spacing at panel edges: 2 in. (50.8 mm) to 6 in. (152 mm).

(d) Height-to-width aspect ratio (h:w): 1:1 to 4:1.

(e) Sheathing screw shall be minimum No. 8.

(f) *Yield strength* of *steel sheet sheathing* shall not be greater than 50 ksi (345 MPa).

See Section B5.2.2.1 for *Type I shear wall* height-to-length aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, for additional requirements.

B5.2.2.3.3 Wood Structural Panel Sheathing

The *nominal strength* [*resistance*] per unit length for assemblies with *wood structural panel* sheathing and *panel blocking* shall be as specified in Table B5.2.2.3-2.

In the United States and Mexico, increases in the *nominal strengths* [*resistances*] in Table B5.2.2.3-2, as allowed by other standards, shall not be permitted.

Tabulated values in Table B5.2.2.3-2 shall be applicable for short-term *load* duration only (wind or seismic *loads*). In the United States and Mexico, for *loads* of normal or permanent load duration as defined by the AWC NDS, the values shall be multiplied by 0.63 (normal) or 0.56 (permanent). In Canada, for *loads* of permanent term *load* duration (dead), the values shall be multiplied by 0.56. For standard term *load* duration (snow and occupancy), the values shall be multiplied by 0.80. For other permanent and standard *load* combinations where the specified dead *load* is greater than the specified standard term *load,* the values shall be multiplied by a factor equal to 0.8 – 0.43 log (D/ST) ≥ 0.56, where D = specified dead *load* and ST = specified standard term *load* based on snow or occupancy *loads* acting alone or in combination.

Systems tabulated in Table B5.2.2.3-2 shall conform to the following additional requirements:

(a) *Wood structural panel* sheathing is manufactured using exterior glue and complies with the following, as applicable:
   (1) In the United States and Mexico, DOC PS 1 or PS 2.
   (2) In Canada, CSA O121, O151 or CAN/CSA O325.0.

(b) *Wood structural panels* are applied either parallel to or perpendicular to *studs*.

(c) Screws used to attach *wood structural panels* are a minimum No. 8 with a minimum head diameter of 0.285 inch (7.24 mm) and are in accordance with ASTM C1513.

B5.2.2.3.4 Gypsum Board Panel Sheathing

The *nominal strength* [*resistance*] per unit length for assemblies with gypsum board panel sheathing and *panel blocking* shall be as specified in Table B5.2.2.3-3.

The *nominal strength* [*resistance*] per unit length for unblocked assemblies shall be as specified in Table B5.2.2.3-3 multiplied by 0.35.

Tabulated values in Table B5.2.2.3-3 shall be applicable for short-term *load* duration only (wind or seismic *loads*).

Systems tabulated in Table B5.2.2.3-3 shall conform to the following additional requirements:

(a) Gypsum board panels comply with ASTM C1396/C1396M.

(b) Gypsum board panels are applied perpendicular to *studs* with *panel blocking* behind the horizontal joint and with *stud blocking* between the first two end *studs*, at each end of the wall, or applied vertically with all edges attached to *structural members*.

(c) Screws used to attach gypsum board are a minimum No. 6 and are in accordance with ASTM C954 or ASTM C1002, as applicable.

B5.2.2.3.5 Fiberboard Panel Sheathing

The *nominal strength* [*resistance*] per unit length for assemblies with *fiberboard* panel sheathing and *panel blocking* shall be as specified in Table B5.2.2.3-4.

Tabulated values in Table B5.2.2.3-4 shall be applicable for short-term *load* duration only (wind *loads* only).

Systems tabulated in Table B5.2.2.3-4 shall conform to the following additional requirements:

(a) *Fiberboard* panels comply with ASTM C208.

(b) *Fiberboard* is applied perpendicular to *studs* with *panel blocking* behind the horizontal joint and with *stud blocking* between the first two end *studs*, at each end of the wall, or applied vertically with all edges attached to *structural members*.

(c) Screws used to attach *fiberboard* are a minimum No. 8 and are in accordance with ASTM C1513. Head style is selected to provide a flat bearing surface in contact with the sheathing with a head diameter not less than 0.43 inches (10.9 mm). Screws are driven so that their flat bearing surface is flush with the surface of the sheathing.

B5.2.2.3.6 Combined Systems

For a *Type I shear wall* sheathed with the same material and fastener spacing on opposite faces of the same wall, the *nominal strength* [*resistance*] of the complete wall based on Tables B5.2.2.3-1 through B5.2.2.3-4 shall be determined by adding the *nominal strength* [*resistance*] from the two opposite faces together.

For a *Type I shear wall* having more than a single sheathing material or fastener spacing, the *nominal strength* [*resistance*] of the complete wall based on Tables B5.2.2.3-1 through B5.2.2.3-4 shall not be permitted to be determined by adding the *nominal strength* [*resistance*] from the different individual walls; rather, it shall be determined in accordance with the following:

(a) For a *Type I shear wall* having more than a single sheathing material or fastener configuration along one face of the same wall line, the *nominal strength* [*resistance*] shall be taken either assuming the weaker (lower *nominal strength* [*resistance*]) material or fastener configuration exists for the entire length of the wall, or the stronger (higher *nominal strength* [*resistance*]) material or fastener configuration exists for its own length, whichever is greater.

(b) In the United States and Mexico, for a *Type I shear wall* sheathed with 15/32 in. structural 1 sheathing (4-ply) or 7/16 in. rated sheathing (OSB) on one side with screw spacing at 6 inches (150 mm) o.c. edge and 12 inches (300 mm) o.c. field and with 1/2 in. gypsum board on the opposite side with screw spacing at 7 in. (178 mm) o.c. edge and 7 in. (178 mm) o.c. field, the *nominal strengths* in Table B5.2.2.3-2 are permitted to be increased by 30 percent.

(c) For other cases of a *Type I shear wall* having more than a single sheathing material or fastener configuration on opposite faces of the wall, the *nominal strength* [*resistance*] shall be taken either assuming the weaker material or fastener configuration exists for both faces of the wall, or the stronger material or fastener configuration exists for its own face alone, whichever is greater.

Table B5.2.2.3-1
Unit Nominal Shear Strength [Resistance] (v_n) [1, 2]
For Shear Walls with Steel Sheet Sheathing on One Side of Wall

United States and Mexico (Pounds Per Foot)								
Sheathing	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (inches)				Stud Blocking Required	Designation Thickness of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		6/12	4/12	3/12	2/12			
0.018” steel sheet	2:1	485	-	-	-	No	33 (min.)	8
0.027” steel sheet	4:1 [3]	-	1,000	1085	1170	No	43 (min.)	8
	4:1 [3]	645	710	780	845	No	33 (min.)	8
0.030” steel sheet	4:1 [3]	795	960	1005	1055	No	33 (min.)	8
	4:1 [3]	910	1015	1040	1070	No	43 (min.)	8
	4:1 [3]	-	-	-	1355	Yes	43 (min.)	10
0.033” steel sheet	4:1 [3]	1035	1145	1225	1300	No	33 (min.)	8
	4:1 [3]	1055	1170	1235	1305	No	43 (min.)	8
	4:1 [3]	-	-	-	1505	Yes	43 (min.)	10
	4:1 [3]	-	-	-	1870	No	54 (min.)	8
	4:1 [3]	-	-	-	2085	Yes	54 (min.)	10
Canada (kN/m)								
Sheathing	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (mm)				Stud Blocking Required	Designation Thickness of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		150/ 300	100/ 300	75/ 300	50/ 300			
0.46 mm steel sheet	2:1	4.1	-	-	-	No	33 (min)	8
	2:1	4.5	6.0	6.8	7.5	No	43 (min)	8
	2:1	7.4	9.7	11.6	13.5	Yes	43 (min)	8
0.68 mm steel sheet	2:1	6.5	7.2	7.9	8.7	No	33 (min)	8
0.76 mm steel sheet	4:1 [3]	8.9	10.6	11.6	12.5	No	43 (min)	8
	2:1	11.7	14.3	-	-	Yes	43 (min)	8
	2:1	-	-	19.9	23.3	Yes	54 (min)	8
0.84 mm steel sheet	4:1 [3]	10.7	12.0	13.0	14.0	No	43 (min)	8

1. For SI: 1” = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
2. See Section B5.2.2.3.6 for requirements for sheathing applied to both sides of wall.
3. See Section B5.2.2.1 for Type I shear wall height-to-length aspect ratios (h/w) greater than 2:1, but not exceeding 4:1.

Table B5.2.2.3-2
Unit Nominal Shear Strength [Resistance] (v_n) [1, 2]
For Shear Walls with Wood Structural Panel Sheathing on One Side of Wall

United States and Mexico (Pounds Per Foot)						
Sheathing	**Maximum Aspect Ratio (h/w)**	**Fastener Spacing at Panel Edges/Field (inches)**				**Designation Thickness of Stud, Track and Blocking (mils)**
		6/12	4/12	3/12	2/12	
15/32” structural 1 (4-ply)	2:1	1065	1410	1735	1910	43 (min.)
7/16” rated sheathing (OSB)	2:1	910	1410	1735	1910	33 (min.)
7/16” rated sheathing (OSB) oriented perpendicular to framing	2:1	1020	-	-	-	33 (min.)
7/16” rated sheathing (OSB)	4:1 [3]	-	1025	1425	1825	33 (min.)

Canada (kN/m)					
Sheathing	**Maximum Aspect Ratio (h/w)**	**Fastener Spacing at Panel Edges/Field (mm)**			**Designation Thickness of Stud, Track and Blocking (mils)**
		150/300	100/300	75/300	
9.5 mm CSP Sheathing	4:1 [3]	8.5	11.8	14.2	43 (min.)
12.5 mm CSP Sheathing	4:1 [3]	9.5	13.0	19.4	43 (min.)
12.5 mm DFP Sheathing	4:1 [3]	11.6	17.2	22.1	43 (min.)
9 mm OSB 2R24/W24	4:1 [3]	9.6	14.3	18.2	43 (min.)
11 mm OSB 1R24/2F16/W24	4:1 [3]	9.9	14.6	18.5	43 (min.)

1. For SI: 1” = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
2. See Section B5.2.2.3.6 for requirements for sheathing applied to both sides of wall.
3. See Section B5.2.2.1 for Type I shear wall height-to-length aspect ratios (h/w) greater than 2:1, but not exceeding 4:1.

Table B5.2.2.3-3
Unit Nominal Shear Strength [Resistance] (v_n) [1, 2]
For Shear Walls with Gypsum Board Panel Sheathing on One Side of Wall

United States and Mexico (Pounds Per Foot)						
Sheathing	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (inches)				Designation Thickness of Stud, Track and Blocking (mils)
		8/12	4/12	7/7	4/4	
½" gypsum board	2:1	230	295	290	425	33 (min)

Canada (kN/m)					
Sheathing	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (mm)			Designation Thickness of Stud, Track and Blocking (mils)
		200/300	150/300	100/300	
12.5 mm gypsum board	2:1	2.7	3.1	3.4	33 (min)

1. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
2. See Section B5.2.2.3.6 for requirements for sheathing applied to both sides of wall.

Table B5.2.2.3-4
Unit Nominal Shear Strength [Resistance] (v_n) [1, 2]
For Shear Walls with Fiberboard Panel Sheathing on One Side of Wall

United States and Mexico (Pounds Per Foot)					
Sheathing	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (inches)			Designation Thickness of Stud, Track and Blocking (mils)
		4/6	3/6	2/6	
½" fiberboard	1:1	425	615	670	33 (min)

Canada (kN/m)					
Sheathing	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (mm)			Designation Thickness of Stud, Track and Blocking (mils)
		100/150	75/150	50/150	
12.5 mm fiberboard	1:1	5.0	7.2	7.8	33 (min)

1. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
2. See Section B5.2.2.3.6 for requirements for sheathing applied to both sides of wall.

B5.2.3 Available Strength [Factored Resistance]

The *available strength* [*factored resistance*] ($\phi_v V_n$ for *LRFD* and *LSD* or V_n/Ω_v for *ASD*) shall be determined from the *nominal strength* [*resistance*] using the applicable *safety factors* (Ω) or *resistance factors* (ϕ) given in this section in accordance with the applicable design method – *ASD*, *LRFD*, or *LSD* as follows:

Ω_v = 2.00 for *ASD*

ϕ_v = 0.65 for *LRFD*

ϕ_v = 0.70 for *LSD* (except gypsum and *fiberboard* sheathed walls)

ϕ_v = 0.60 for *LSD* (gypsum and *fiberboard* sheathed walls)

B5.2.4 Collectors and Anchorage

Design of *collectors* and anchorage for *shear walls* shall conform to the requirements of this section.

B5.2.4.1 Collectors and Anchorage for In-Plane Shear

Collectors and anchorage for in-plane shear shall be designed in accordance with this section.

B5.2.4.1.1 Type I Shear Walls

The unit shear force, v, transmitted into the top and out of the base of the *Type I shear wall* shall be determined in accordance with the following:

$$v = \frac{V}{L} \qquad (Eq.\ B5.2.4\text{-}1)$$

where

v = Unit shear force, plf (kN/m)

V = Shear force determined in accordance with applicable *ASD*, *LRFD* or *LSD* *load combinations* in *Type I shear wall*, lbs (kN)

L = Length of *Type I shear wall*, feet (m)

B5.2.4.1.2 Type II Shear Walls

The unit shear force, v, transmitted into the top and out of the base of the *Type II shear wall segments*, and into *collectors* connecting *Type II shear wall segments*, shall be determined in accordance with the following:

$$v = \frac{V}{C_a \sum L_i} \qquad (Eq.\ B5.2.4\text{-}2)$$

where

v = Unit shear force, plf (kN/m)

V = Shear force determined in accordance with applicable *ASD*, *LRFD* or *LSD* *load combinations* in *Type II shear wall*, lbs (kN)

C_a = Shear resistance adjustment factor from Table B5.2.2.2-1

ΣL_i = Sum of lengths of *Type II shear wall segments*, feet (m)

B5.2.4.2 Uplift Anchorage at Wall Ends

Uplift anchorage at *shear wall* ends shall be designed in accordance with this section.

B5.2.4.2.1 General

Where uplift anchorage resists the overturning load from the story or stories above, the anchorage shall be sized for the anchorage force resulting from the lateral forces at its level plus the story or stories above.

Hold-downs shall be designed to resist the sum of both *shear wall* overturning and roof uplift, as applicable.

Uplift anchorage forces are permitted to be reduced to account for dead load in accordance with the *applicable building code*.

B5.2.4.2.2 Type I Shear Walls

Anchorage for uplift forces due to overturning shall be provided at each end of the *Type I shear wall*. The uplift anchorage force contributed from each level shall be determined in accordance with the following:

$$C = \frac{Vh}{L} \qquad (Eq.\ B5.2.4\text{-}3)$$

where

C = Uplift anchorage force contributed from each level, lbs (kN)

V = Shear force determined in accordance with applicable *ASD*, *LRFD* or *LSD load combinations* in *Type I shear wall*, lbs (kN)

h = *Shear wall* height, feet (m)

L = Length of *Type I shear wall* including anchor offsets, feet (m)

B5.2.4.2.3 Type II Shear Walls

Anchorage for uplift forces due to overturning shall be provided at each end of the *Type II shear wall*. For each level, the uplift anchorage force contributed shall be determined in accordance with the following:

$$C = \frac{Vh}{C_a \sum L_i} \qquad (Eq.\ B5.2.4\text{-}4)$$

where

C = Uplift anchorage force contributed from each level, lbs (kN)

V = Shear force determined in accordance with applicable *ASD*, *LRFD* or *LSD load combinations* in *Type II shear wall*, lbs (kN)

h = *Shear wall* height, feet (m)

C_a = Shear resistance adjustment factor from Table B5.2.2.2-1

ΣL_i = Sum of lengths of *Type II shear wall segments*, feet (m)

B5.2.4.3 Uplift Anchorage Between Type II Shear Wall Ends

In addition to the requirements of Section B5.2.4.2, *Type II shear wall* bottom plates at full-height sheathing locations shall be anchored for a uniform uplift force equal to the unit shear force, v, determined in accordance with Section B5.2.4.1 plus any applicable roof uplift forces.

B5.2.5 Design Deflection

The deflection of a blocked *Type I shear wall* with *steel sheet sheathing* or *wood structural panel* sheathing shown in Tables B5.2.2.3-1 and B5.2.2.3-2 is permitted to be calculated in accordance with the following:

$$\delta = \frac{2vh^3}{3E_sA_cb} + \omega_1\omega_2\frac{vh}{\rho Gt_{sheathing}} + \omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v \qquad (Eq.\ B5.2.5\text{-}1)$$

where

A_c = Gross cross-sectional area of *chord member*, in square inches (mm^2)

b = Length of the *shear wall*, in inches (mm)

E_s = Modulus of elasticity of steel
= 29,500,000 psi (203,000 MPa)

G = Shear modulus of sheathing material, in pounds per square inch (MPa)

h = Wall height, in inches (mm)

s = Maximum fastener spacing at panel edges, in inches (mm)

$t_{sheathing}$ = Nominal panel thickness, in inches (mm)

t_{stud} = Framing *designation thickness*, in inches (mm)

v = Shear demand, in pounds per linear inch (N/mm)
= V/b (*Eq*. B5.2.5-2)

V = Total lateral *load* applied to the *shear wall*, in pounds (N)

β = 67.5 for plywood other than Canadian Soft Plywood (CSP),
= 55 for OSB and CSP for U.S. Customary units ($lb/in^{1.5}$)
= 2.35 for plywood other than CSP,
= 1.91 for OSB and CSP for SI units ($N/mm^{1.5}$)
= $29.12(t_{sheathing}/0.018)$ for sheet steel (for $t_{sheathing}$ in inches) ($lb/in^{1.5}$) (*Eq*. B5.2.5-3a)
= $1.01(t_{sheathing}/0.457)$ for sheet steel (for $t_{sheathing}$ in mm) ($N/mm^{1.5}$) (*Eq*. B5.2.5-3b)

δ = Calculated deflection, in inches (mm)

δ_v = Vertical deformation of anchorage/attachment details, in inches (mm)

ρ = 1.85 for plywood other than CSP, 1.05 for OSB and CSP
= $0.075(t_{sheathing}/0.018)$ for sheet steel (for $t_{sheathing}$ in inches) (*Eq*. B5.2.5-4a)
= $0.075(t_{sheathing}/0.457)$ for sheet steel (for $t_{sheathing}$ in mm) (*Eq*. B5.2.5-4b)

ω_1 = s/6 (for s in inches) (*Eq*. B5.2.5-5a)
= s/152.4 (for s in mm) (*Eq*. B5.2.5-5b)

ω_2 = $0.033/t_{stud}$ (for t_{stud} in inches) (*Eq*. B5.2.5-6a)
= $0.838/t_{stud}$ (for t_{stud} in mm) (*Eq*. B5.2.5-6b)

ω_3 = $\sqrt{\frac{(h/b)}{2}}$ (*Eq*. B5.2.5-7)

ω_4 = 1 for wood structural panels
= $\sqrt{\frac{33}{F_y}}$ for sheet steel (for F_y in ksi) (*Eq*. B5.2.5-8a)
= $\sqrt{\frac{227.5}{F_y}}$ for sheet steel (for F_y in MPa) (*Eq*. B5.2.5-8b)

The deflection of a *Type I shear wall* with gypsum panel sheathing or *fiberboard* sheathing or *Type II shear wall* shall be determined by principles of mechanics considering the deformation of the sheathing and its attachment, *chord studs* and *hold-downs*.

B5.3 Strap Braced Wall Design

Strap braced walls shall be designed in accordance with the requirements of this section and shall be installed in accordance with the requirements of Section C3.6.2.

User Note:
See Section A1.2.2 for applicability of this section to cold-formed steel structural members and connections in seismic force-resisting systems.

B5.3.1 General

S*trap braced walls* with height-to-length aspect ratio (h:L) less than 2:1 are permitted to be designed without consideration of *chord stud* end moments due to joint fixity.

B5.3.1.1 For *strap braced walls* where the height-to-length ratio *(*h:L) is greater than or equal to 2:1:

(1) An analysis of the *strap braced wall* based on the assumption of full *joint* fixity shall be used to determine the member and *connection* forces.

(2) In considering the moment along the length of the *chord stud,* locations that are stiffened by a *hold-down* need not be checked for combined axial and bending load effects.

User Note:
Commentary Section B5.3.1 provides expressions for a frame analysis with full joint fixity. The purpose of the frame analysis is to determine the moment demand on the *chord studs*.

From the frame analysis, the *chord stud* is designed for combined axial and bending load effects in combination with all other applicable load effects, in accordance with Section B5.3.2.

B5.3.2 Nominal Strength [Resistance]

The *nominal strength* [*resistance*] for the *strap*, *connections*, and *chord studs* shall be determined in accordance with AISI S100.

B5.3.3 Available Strength [Factored Resistance]

The *available strength* [*factored resistance*] for the *strap*, *connections*, and *chord studs* shall be determined in accordance with AISI S100.

B5.3.4 Collectors and Anchorage

Design of *collectors* and anchorage for *strap braced walls* shall conform to the requirements of this section.

B5.3.4.1 Collectors and Anchorage for In-Plane Shear

Collectors and anchorage shall be designed to transmit the shear force into the top and out of the base of the *strap braced wall*.

B5.3.4.2 Uplift Anchorage at Wall Ends

Where uplift anchorage resists the overturning load from the story or stories above, the anchorage shall be sized for the anchorage force at its level plus the anchorage force of the story or stories above.

Hold-downs shall be designed to resist the sum of both *strap braced wall* overturning and roof uplift, as applicable.

Uplift anchorage forces are permitted to be reduced to account for dead load in accordance with the *applicable building code*.

Anchorage for uplift forces due to overturning shall be provided at each end of the *strap braced wall*. Uplift anchorage and boundary chord forces shall be determined in accordance with the following:

$$C = \frac{Vh}{L} \qquad (Eq.\ B5.3.4.2\text{-}1)$$

where

C = Boundary chord force (tension/compression), lbs (kN)

V = Shear force determined in accordance with applicable *ASD*, *LRFD* or *LSD load combinations* in *strap braced wall*, lbs (kN)

h = *Strap braced wall* height, feet (m)

L = Length of *strap braced wall* including anchor offsets, feet (m)

B5.3.5 Design Deflection

The deflection of a *strap braced wall* shall be determined by principles of mechanics considering the deformation of the *strap*, *chord studs* and *hold-downs*.

B5.4 Diaphragm Design

Diaphragms shall be designed in accordance with Section B5.4.1 or B5.4.2, as applicable.

B5.4.1 Cold-Formed Steel Sheathed Diaphragms

Diaphragms sheathed with *cold-formed steel* shall be designed based on principles of mechanics in accordance with Section B1.2.6 and the provisions of this section, as applicable.

B5.4.1.1 Non-Composite Steel Floor Deck Diaphragms

In the United States and Mexico, *non-composite steel floor decks* shall be designed in accordance with AISI S100. *S*hear strength and stiffness of *diaphragms* sheathed with *non-composite steel floor deck* shall be in accordance with AISI S310.

In Canada, *non-composite steel floor decks* shall be designed in accordance with CSA S136. *S*hear resistance and stiffness of *diaphragms* sheathed with *non-composite steel floor deck* shall be in accordance with AISI S310.

User Note:

In the United States, IBC Chapter 22 adopts ANSI/SDI-NC for the design of non-composite steel floor deck.

In Canada, it is common practice for the design of *non-composite steel floor deck* to be designed following the recommendations of CSSBI 12M.

Shear strength and stiffness tables for *diaphragms* sheathed with *non-composite steel floor deck* following the AISI S310 methods can be found in the SDI *Steel Deck on Cold-Formed Steel Framing Design Manual*, SDI *Diaphragm Design Manual*, product approval agency evaluation reports, and manufacturer catalogs.

References:

CSSBI 12M-2017, *Standard for Composite Steel Deck*, Canadian Sheet Steel Building Institute, 2017.
IBC (2018) *International Building Code*, International Code Council, 2018.

SDI (2018) *Steel Deck on Cold-Formed Steel Framing Design Manual 1st Edition*, Steel Deck Institute, 2018.

SDI (2015) *Diaphragm Design Manual 4th Edition*, Steel Deck Institute, 2015.

ANSI/SDI NC-2017, *Standard for Non-Composite Steel Floor Deck*, Steel Deck Institute, 2017.

B5.4.1.2 Steel Roof Deck Diaphragms

In the United States and Mexico, *steel roof deck* shall be designed in accordance with AISI S100. Shear strength and stiffness of *diaphragms* sheathed with *steel roof deck* shall be in accordance with AISI S310.

In Canada, *steel roof decks* shall be designed in accordance with CSA S136. Shear resistance and stiffness of *diaphragms* sheathed with *steel roof deck* shall be in accordance with AISI S310.

User Note:
In the United States, IBC Chapter 22 adopts ANSI/SDI RD for the design of *steel roof deck*.

In Canada, it is common practice for *steel roof deck* to be designed following the recommendations of CSSBI 10M.

Shear strength and stiffness tables for *diaphragms* sheathed with *steel roof deck* following the AISI S310 methods can be found in the SDI *Steel Deck on Cold-Formed Steel Framing Design Manual*, SDI *Diaphragm Design Manual*, product approval agency evaluation reports, and manufacturer catalogs.

References:

CSSBI 10M-2013, *Standard for Steel Roof Deck*, Canadian Sheet Steel Building Institute, 2013.

IBC (2018) *International Building Code*, International Code Council, 2018.

SDI (2018), *Steel Deck on Cold-Formed Steel Framing Design Manual 1ST Edition*, Steel Deck Institute, 2018.

SDI (2015), *Diaphragm Design Manual 4th Edition*, Steel Deck Institute, 2015.

ANSI/SDI RD-2017, *Standard for Steel Roof Deck*, Steel Deck Institute, 2017.

B5.4.1.3 Composite Steel Deck-Slab Diaphragms

In the United States and Mexico, *composite steel deck* shall be designed and constructed in accordance with AISI S100. The *diaphragm* shear strength and stiffness of *composite steel deck-slabs* shall be in accordance with AISI S310.

In Canada, *composite steel deck* shall be designed and constructed in accordance with CSA S136. *Diaphragm* shear strength and stiffness of c*omposite steel deck-slabs* shall be in accordance with AISI S310.

User Note:
In the United States, IBC Chapter 22 adopts ANSI/SDI-C for the design of *composite steel deck-slabs*.

In Canada, it is common practice for *composite steel deck-slabs* to be designed following the recommendations of CSSBI 12M.

Diaphragm shear strength and stiffness tables for *composite steel deck-slabs* following the AISI S310 methods can be found in the SDI *Diaphragm Design Manual*, product approval agency evaluation reports, and manufacturer catalogs.

References:

CSSBI 12M-2017, Standard for Composite Steel Deck, Canadian Sheet Steel Building Institute, 2017.

IBC (2018), International Building Code, International Code Council, 2018.

SDI (2015), Diaphragm Design Manual 4th Edition, Steel Deck Institute, 2015.

ANSI/SDI C-2017, Standard for Composite Steel Floor Deck – Slabs, Steel Deck Institute, 2017.

B5.4.1.4 Steel Panel Roof System Diaphragms

Steel panel roof systems shall be designed in accordance with AISI S100 [CSA S136]. The *diaphragm* shear strength and stiffness of *steel panel roof systems* shall be in accordance with AISI S310. *Steel panel roof systems* shall also meet the *applicable building code* provisions for structural metal panel roof systems.

User Note:

In the United States, provisions for structural metal panel roof systems, including *steel panel roof systems*, are found in IBC Section 1504.3.2.

In Canada, provisions for structural metal panel roof systems, including *steel panel roof systems*, are found in NBC section 9.26.13 Sheet Metal Roofing.

Reference:

IBC (2018), *International Building Code*, International Code Council, 2018.

B5.4.1.5 Other Cold-Formed Steel Sheathed Diaphragms

Cold-formed steel sheathed *diaphragms* other than those specified in Sections B5.4.1.1, B5.4.1.2, B5.4.1.3, or B5.4.1.4 shall be designed or tested, as applicable, in accordance with AISI S100 [CSA S136].

B5.4.2 Wood Structural Panel Sheathed Diaphragm

Diaphragms sheathed with *wood structural panels* shall be designed based on principles of mechanics in accordance with Section B1.2.6 or tests in accordance with Section B5.4.2.5. Alternatively, in the United States and Mexico, *diaphragms* sheathed with *wood structural panels* are permitted to be designed as either blocked *diaphragms* or unblocked *diaphragms* in accordance with the requirements of Sections B5.4.2.1 through B5.4.2.4 and shall be installed in accordance with the requirements of Section C3.6.3.

B5.4.2.1 General

Diaphragms shall conform to the following requirements:

(a) The aspect ratio (length/width) of the *diaphragm* does not exceed 4:1 for blocked *diaphragms* and 3:1 for unblocked *diaphragms*.

(b) *Joists* and *track* are ASTM A1003 Structural Grade 33 (Grade 230) Type H steel for members with a *designation thickness* of 33 and 43 mils, and ASTM A1003 Structural Grade 50 (Grade 340) Type H steel for members with a *designation thickness* equal to or greater than 54 mils.

(c) The minimum *designation thickness* of *structural members* is 33 mils.

(d) *Joists* are *C-shape* members with a minimum *flange* width of 1-5/8 in. (41.3 mm), minimum *web* depth of 3-1/2 in. (89 mm) and minimum *edge stiffener* of 3/8 in. (9.5

mm).

(e) *Track* has a minimum *flange* width of 1-1/4 in. (31.8 mm) and a minimum *web* depth of 3-1/2 in. (89 mm).

(f) Screws for *structural members* are a minimum No. 8 and are in accordance with ASTM C1513.

(g) *Wood structural panel* sheathing is manufactured using exterior glue and complies with DOC PS 1 or PS 2.

(h) Screws used to attach *wood structural panels* are minimum No. 8 where *structural members* have a *designation thickness* of 54 mils or less and No. 10 where *structural members* have a *designation thickness* greater than 54 mils and are in accordance with ASTM C1513.

(i) Screws in the field of the panel are attached to intermediate supports at a maximum 12-inch (305 mm) spacing along the *structural members*.

(j) Panels less than 12 inches (305 mm) wide are not used.

(k) Maximum *joist* spacing is 24 inches (610 mm) on center.

(l) Where *diaphragms* are designed as blocked, all panel edges are attached to *structural members* or *panel blocking*.

(m) Where used as *panel blocking*, flat *strap* is a minimum thickness of 33 mils with a minimum width of 1-1/2 inches (38.1 mm).

B5.4.2.2 Nominal Strength

The *nominal strength* per unit length for blocked and unblocked assemblies shall be as specified in Table B5.4.2.2-1.

Tabulated values in Table B5.4.2.2-1 are applicable for short-term *load* duration only (wind or seismic *loads*). For *loads* of normal or permanent *load* duration as defined by the AWC NDS, the values shall be multiplied by 0.75 (normal) or 0.67 (permanent).

Table B5.4.2.2-1
Nominal Shear Strength (R_n) per Unit Length for Diaphragms With Wood Structural Panel Sheathing[1]

United States and Mexico (Pounds Per Foot)							
Sheathing	Thickness (in.)	Blocked				Unblocked	
		Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.)				Screws spaced maximum of 6 in. on all supported edges	
		6	4	2.5	2	Load perpendicular to unblocked edges and continuous panel joints	All other configurations
		Screw spacing at all other panel edges (in.)					
		6	6	4	3		
Structural I	3/8	768	1022	1660	2045	685	510
	7/16	768	1127	1800	2255	755	565
	15/32	925	1232	1970	2465	825	615
C-D, C-C and other graded wood structural panels	3/8	690	920	1470	1840	615	460
	7/16	760	1015	1620	2030	680	505
	15/32	832	1110	1770	2215	740	555

1. For SI: 1 in. = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.

B5.4.2.3 Available Strength

The *available strength* ($\phi_v V_n$ for *LRFD* or V_n/Ω_v for *ASD*) shall be determined from the *nominal strength* using the applicable *safety factors* (Ω) or *resistance factors* (ϕ) given in this section in accordance with the applicable design method —*ASD* or *LRFD,* as follows:

Ω_v = 2.00 (*ASD*)

ϕ_v = 0.65 (*LRFD*)

B5.4.2.4 Design Deflection

The deflection of a blocked *diaphragm* with *wood structural panel* sheathing shown in Table B5.4.2.2-1 is permitted to be calculated in accordance with Section B5.4.2.4.1. The deflection of an unblocked *diaphragm* with *wood structural panel* sheathing shown in Table B5.4.2.2-1 is permitted to be calculated in accordance with Section B5.4.2.4.2.

B5.4.2.4.1 Blocked Diaphragms

The deflection of a blocked *diaphragm*, δ_b, is permitted to be determined in accordance with the following:

$$\delta_b = \frac{0.052vL^3}{E_s A_c b} + \omega_1\omega_2 \frac{vL}{\rho G t_{sheathing}} + \omega_1^{5/4}\omega_2(\alpha)\left(\frac{v}{2\beta}\right)^2 + \frac{\sum_{j=1}^{n}\Delta_{ci}X_i}{2b} \quad (Eq.\ B5.4.2.4\text{-}1)$$

where

A_c = Gross cross-sectional area of *chord member*, in square inches (mm^2)
b = *Diaphragm* depth parallel to direction of *load*, in inches (mm)
E_s = Modulus of elasticity of steel
= 29,500,000 psi (203,000 MPa)
G = Shear modulus of sheathing material, in pounds per square inch (MPa)
L = *Diaphragm* length perpendicular to direction of *load*, in inches (mm)
n = Number of *chord* splices in *diaphragm* (considering both *diaphragm chords*)
s = Maximum fastener spacing at panel edges, in inches (mm)
$t_{sheathing}$ = Nominal panel thickness, in inches (mm)
t_{stud} = Nominal framing thickness, in inches (mm)
v = Shear demand, in pounds per linear inch (N/mm)
= V/(2b) (*Eq*. B5.4.2.4-2)
V = Total lateral *load* applied to the *diaphragm*, in pounds (N)
X_i = Distance between the "ith" *chord*-splice and the nearest support, in inches (mm)
α = Ratio of the average *load* per fastener based on a non-uniform fastener pattern to the average *load* per fastener based on a uniform fastener pattern (= 1 for a uniformly fastened *diaphragm*)
β = 67.5 for plywood other than Canadian Soft Plywood (CSP)
= 55 for OSB and CSP for U.S. Customary units ($lb/in^{1.5}$)
= 2.35 for plywood other than CSP
= 1.91 for OSB for SI units ($N/mm^{1.5}$)
δ_b = Calculated deflection, in inches (mm)
Δ_{ci} = Deformation value associated with "ith" *chord* splice, in inches (mm)
ρ = 1.85 for plywood other than CSP
= 1.05 for OSB and CSP
ω_1 = s/6 (for s in inches) (*Eq*. B5.4.2.4-3a)
= s/152.4 (for s in mm) (*Eq*. B5.4.2.4-3b)
ω_2 = $0.033/t_{stud}$ (for t_{stud} in inches) (*Eq*. B5.4.2.4-4a)
= $0.838/t_{stud}$ (for t_{stud} in mm) (*Eq*. B5.4.2.4-4b)

B5.4.2.4.2 Unblocked Diaphragms

The deflection of an unblocked *diaphragm*, δ_{ub}, is permitted to be determined in accordance with the following:

$\delta_{ub} = 2.50\ \delta_b$ (*Eq*. B5.4.2.4-5)

where

δ_b = Deflection of blocked *diaphragm* determined in accordance with Eq. B5.4.2.4-1

B5.4.2.5 Beam Diaphragm Tests for Non-Steel Sheathed Assemblies

For buildings with the maximum aspect ratio of 4:1 and having *cold-formed steel* roof or floor assemblies having non-steel sheathing, the in-plane *diaphragm nominal shear strength* [*resistance*], S_n, shall be in accordance with Table B5.4.2.2-1 where applicable or shall be established by test in accordance with ASTM E455 and Section K2.1(a) of AISI S100 [CSA S136].

When the failure mode of the assembly is identified to be in the *cold-formed steel* members or sheathing-to-steel fasteners, *safety factors* shall not be less than and *resistance factors* shall not be greater than those prescribed by Section B5.2.3 when determined in accordance with the procedures of Section K2.1 of AISI S100 [CSA S136] with the following definitions of the variables:

β_o = Target reliability index
= 2.5 for United States and Mexico and
= 3.0 for Canada
F_m = Mean value of the fabrication factor
= 1.0
M_m = Mean value of the material factor
= 1.1
V_M = Coefficient of variation of material factor
= 0.10
V_F = Coefficient of variation of fabrication factor
= 0.10
V_Q = Coefficient of variation of the load effect
= 0.21
V_P = Actual calculated coefficient of variation of the test results, without limit
n = Number of connections in the assembly with the same tributary area

When the failure mode of the assembly is identified to be in the sheathing material, a *safety factor* of 2.8 and *resistance factor* of 0.6 shall be used.

C. INSTALLATION

C1 General

Structural members and *connections* shall be installed in accordance with the requirements of this section, as applicable.

C2 Material Condition

The following requirements shall apply to *structural members*, *connectors*, *hold-downs* and mechanical fasteners:

(a) *Structural members*, *connectors*, *hold-downs* and mechanical fasteners shall be as specified by the *approved construction documents*.

(b) Damaged *structural members*, *connectors*, *hold-downs* and mechanical fasteners shall be replaced or repaired in accordance with an *approved* design or as specified by a *registered design professional*.

C2.1 Web Holes

Holes in *webs* of framing members shall be in conformance with AISI S100 [CSA S136], the *approved construction documents*, or shall be reinforced or patched in accordance with an *approved* design as specified by a *registered design professional*.

C2.2 Cutting and Patching

C2.2.1 All cutting of framing members shall be done by sawing, abrasive cutting, shearing, plasma cutting or other *approved* methods acceptable to the *registered design professional*.

C2.2.2 Cutting or notching of *structural members*, including *flanges* and *lips* of *joists*, *studs*, *headers*, *rafters*, and *ceiling joists*, shall be permitted with an *approved* design or as specified by a *registered design professional*.

C2.2.3 Patching of cuts and notches shall be permitted with an *approved* design or as specified by a *registered design professional*.

C2.3 Splicing

Splicing of *joists*, *studs* and other *structural members* shall be permitted with an *approved* design or as specified by a *registered design professional*.

C3 Structural Framing

Structural members shall be installed in accordance with the requirements of this section, as applicable.

C3.1 Foundation

The foundation shall be level and free from defects beneath structural walls. If the foundation is not level, provisions shall be made to provide a uniform bearing surface with a maximum 1/4 in. (6.4 mm) gap between the bottom *track* or rim *track* and the foundation. This shall be accomplished through the use of load bearing shims or grout provided between the underside of the wall bottom *track* or *rim track* and the top of the foundation wall or slab at *stud* or *joist* locations.

C3.2 Ground Contact

Framing shall not be in direct contact with the ground unless permitted by an *approved* design. Framing not in direct contact with the ground shall be installed at a height above the ground in accordance with the *applicable building code*.

C3.3 Floors

C3.3.1 Plumbness and Levelness

Floor joists and floor *trusses* shall be installed plumb and level, except where designed as sloping members.

C3.3.2 Alignment

Floor joists and floor *trusses* shall comply with the in-line framing requirements of Section B1.2.3 and floor sheathing span capacity requirements of Section B1.2.4, as applicable.

C3.3.3 Bearing Width

Floor joists and floor *trusses* shall be installed with full bearing over the width of the bearing wall beneath, a minimum 1-1/2 inch (38 mm) bearing end, or in accordance with an *approved* design or as specified by a *registered design professional*.

C3.3.4 Web Separation

Floor joist webs shall not be in direct contact with *rim track webs*.

C3.4 Walls

C3.4.1 Straightness, Plumbness and Levelness

Wall *studs* shall be installed plumb and walls shall be level in accordance with ASTM C1007, except where designed as sloping members.

C3.4.2 Alignment

Structural wall *studs* shall comply with the in-line framing requirements of Section B1.2.3 and wall sheathing span capacity requirements of Section B1.2.4, as applicable.

C3.4.3 Stud-to-Track Connection

The *connection* between the *stud flange* and the *track flange* shall meet the requirements of Section B3.2.5 and shall permit both ends of the wall *stud* to be connected to the *track* to restrain rotation about the longitudinal wall *stud* axis and horizontal displacement perpendicular to the wall *stud* axis. Further, the maximum gap between the end of the *stud* and the *track web* shall also comply with the following, as applicable:

(a) Ends of axial *load* bearing wall *studs* 54 mils and less shall have square end cuts and shall be seated tight against the *track* with a gap that does not exceed 1/8 in. (3.2 mm) between the end of the wall framing member and the *web* of the *track*. For thicknesses of the *stud* or *track* greater than 54 mil (0.054 inches (1.37mm)), the maximum end gap shall be specified by the *registered design professional*.

(b) Ends of *curtain wall studs* shall have square end cuts and shall be seated in the *track* with a gap that does not exceed 1/4 in. (6.4 mm) between the end of the wall *stud* and the *web* of the *track*, unless otherwise accepted by a *registered design professional*.

C3.4.4 Back-to-Back and Box Headers

Back-to-back and box *headers* designed in accordance with this Standard shall be installed in accordance with Figures C3.4.4-1 and C3.4.4-2, respectively. For box *headers*, it is permitted to connect *track flanges* to the *webs* of *C-shape* sections using 1-inch (25.4 mm) fillet welds spaced at 24 inches (610 mm) on center in lieu of No. 8 screws.

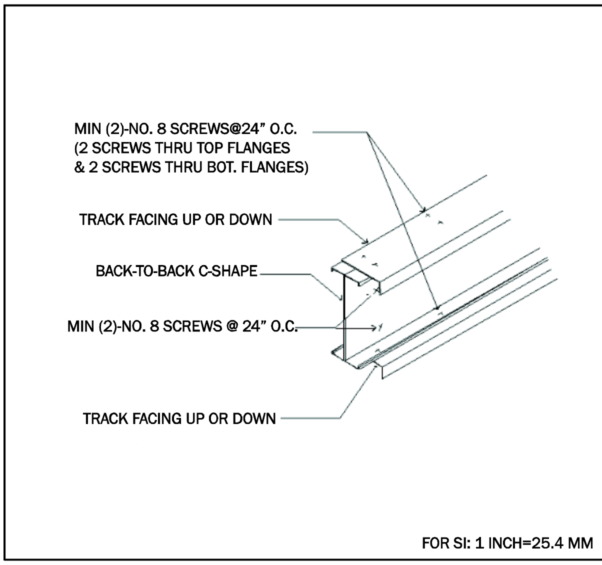

Figure C3.4.4-1 Back-to-Back Header

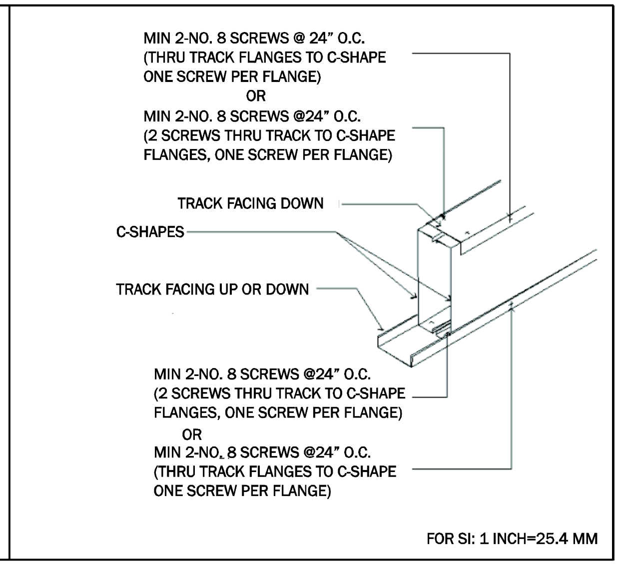

Figure C3.4.4-2 Box Header

C3.4.5 Double and Single L-Headers

Double and single L-*headers* designed in accordance with this Standard shall be installed in accordance with Figures C3.4.5-1 and C3.4.5-2, respectively.

C3.4.6 Inverted L-Header Assemblies

Inverted double or single L-*headers* designed in accordance with this Standard shall be installed in accordance with the following:

(1) The horizontal leg of the inverted L-*header* shall be coped to permit the vertical leg to lap over at least one bearing *stud* at each end. The horizontal leg after coping shall be within 1/2 inch (12.7 mm) of the bearing *stud* at each end.

(2) The horizontal leg of the inverted L-*header* shall be attached to the head *track* at each end and at 12 inches (304.8 mm) on center with minimum #8 screws.

(3) The vertical leg of the inverted L-*header* shall be attached to at least one bearing *stud* at each end and each cripple *stud* with a minimum No. 8 screw top and bottom. The top screw in the vertical leg of the inverted L-*header* shall be located not more than 1 inch (25.4 mm) from the top edge of the vertical leg.

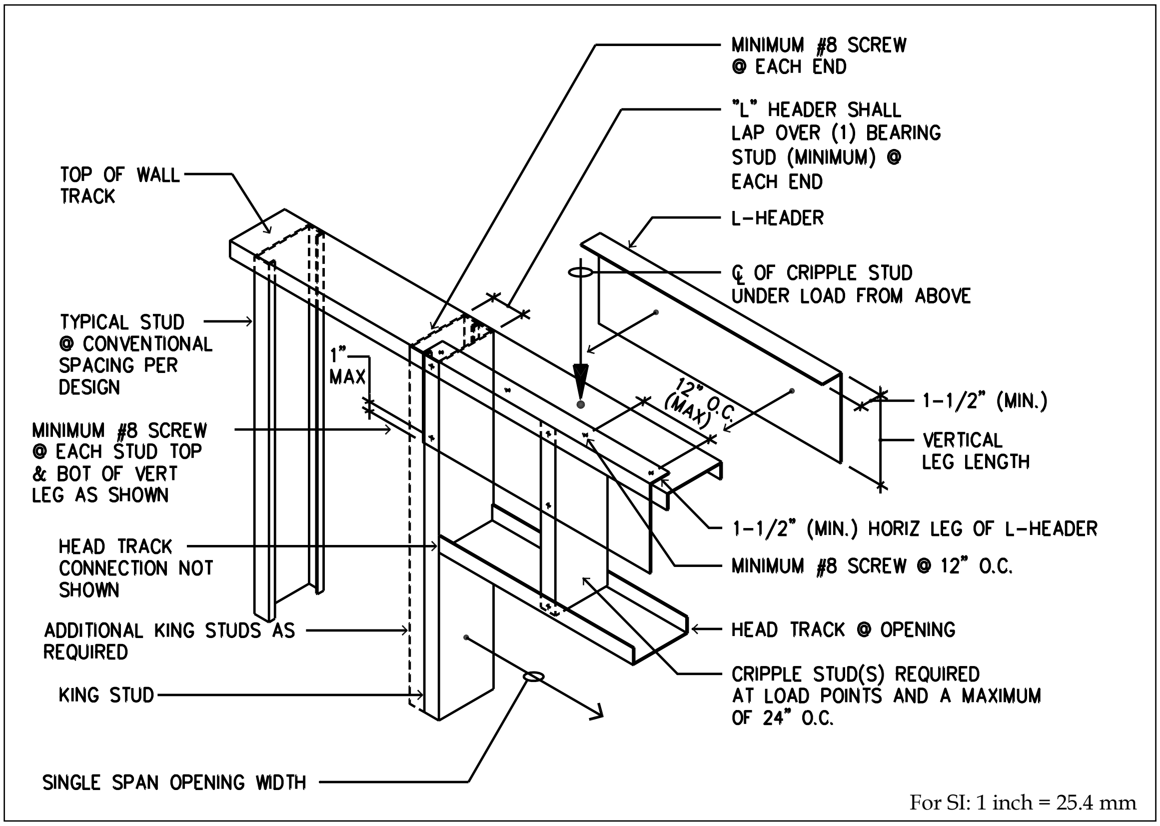

Figure C3.4.5-1 Double L-Header

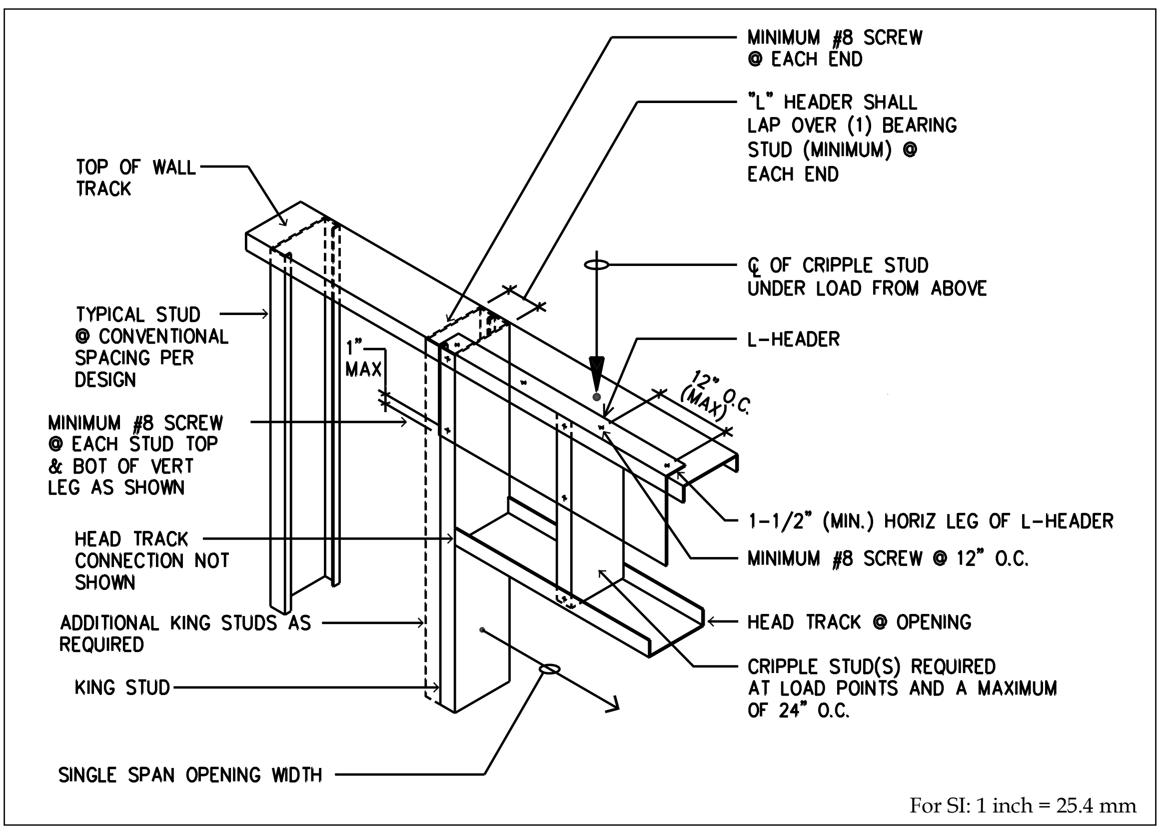

Figure C3.4.5-2 Single L-Header

C3.5 Roofs and Ceilings

C3.5.1 Plumbness and Levelness

Roof and ceiling framing members shall be installed plumb and level, except where designed as sloping members.

C3.5.2 Alignment

Roof and ceiling framing members shall comply with the in-line framing requirements of Section B1.2.3 and roof sheathing span capacity requirements of Section B1.2.4, as applicable.

C3.5.3 End Bearing

Ceiling joists and roof *trusses* shall be installed with full bearing over the width of the bearing wall beneath, or a minimum 1-1/2 in. (38 mm) bearing end condition, or in accordance with an *approved* design or as specified by a *registered design professional*.

C3.6 Lateral Force-Resisting Systems

C3.6.1 Shear Walls

The installation of *shear walls* designed in accordance with this Standard shall conform to the following requirements:

(a) Fasteners along the edges in shear panels are placed from panel edges not less than the following, as applicable:
   (1) In the United States and Mexico, 3/8 in. (9.5 mm).
   (2) In Canada, 12.5 mm (1/2 in.).

(b) Where used as *panel blocking* with *steel sheet sheathing*, flat *strap* is installed on top of or below the sheathing. Where used as *panel blocking* with other than *steel sheet sheathing*, flat *strap* is installed below the sheathing.

(c) Where *panel blocking* is used with other than *steel sheet sheathing*, the screws are installed through the sheathing to the *panel blocking*.

C3.6.2 Strap Braced Walls

No additional provisions.

C3.6.3 Diaphragms

C3.6.3.1 Cold-Formed Steel Sheathed Diaphragms

The installation of *diaphragms* sheathed with *cold-formed steel* shall be in accordance with provisions of the *applicable building code*, an *approved* design or as specified by a *registered design professional*.

C3.6.3.2 Wood Structural Panel Sheathed Diaphragms

The installation of *diaphragms* sheathed with *wood structural panels* and designed in accordance with this Standard shall conform to the following requirements:

(a) Fasteners along the edges in shear panels are placed from panel edges not less than 3/8 in. (9.5 mm).

(b) Where used as *panel blocking*, flat *strap* is installed below the sheathing.
(c) Where *panel blocking* is used, the screws are installed through the sheathing to the *panel blocking*.

C4 Connections

Connections shall be installed in accordance with the requirements of this section, as applicable.

C4.1 Screw Connections

C4.1.1 Steel-to-Steel Screws

Use of a larger-than-specified screw size is permitted if the installation is in accordance with the minimum design spacing and edge distance.

C4.1.2 Installation

C4.1.2.1 Screw fasteners shall extend through the steel *connection* with a minimum of three (3) exposed threads.

C4.1.2.2 Screw fasteners shall penetrate individual components of *connections* without causing permanent separation between components.

C4.1.3 Stripped Screws

C4.1.3.1 Stripped screw fasteners in direct tension shall not be considered effective.

C4.1.3.2 Stripped screw fasteners in shear shall only be considered effective when the number of stripped screw fasteners considered effective does not exceed 25% of the total number of screw fasteners considered effective in the *connection*.

C4.1.4 Overdriven Screws in Shear Walls and Diaphragms Sheathed With Wood Structural Panels

Designation of overdriven screw fasteners in *shear walls* and *diaphragms* sheathed with *wood structural panels* shall be in accordance with Section C4.1.4.1. Remediation of overdriven screw fasteners shall be made in accordance with Section C4.1.4.2. Where all overdriven screws are not remediated, the calculated *nominal strength* [*nominal resistance*] shall be determined in accordance with Section C4.1.4.3.

C4.1.4.1 A screw fastener shall be deemed overdriven where the flat outer surface of the head is driven below the surface of the panel beyond 1/8 in. (3.18 mm) for a nominal panel thickness of 7/16 in. (11.1 mm) or greater, or beyond 1/16 in. (1.59 mm) for a nominal panel thickness less than 7/16 in. (11.1 mm).

C4.1.4.2 Overdriven screw fasteners are permitted to be remediated by removal and replacement with a screw with a larger head diameter. Unscrewing an overdriven screw fastener until it is no longer considered overdriven in accordance with Section C4.1.4.1 is not permitted.

C4.1.4.3 For *shear walls* or *diaphragms* with overdriven screw fasteners, the calculated *nominal strength* [*nominal resistance*] shall be determined in accordance with either Section C4.1.4.3.1 or Section C4.1.4.3.2, as applicable.

C4.1.4.3.1 No strength reduction for overdriven fasteners is required where all the following criteria are met:

(1) In any of the four corners of an individual *wood structural panel*, none of the three fasteners closest to a panel's corner are overdriven;
(2) No more than 10% of fasteners along the perimeter, excluding the panel corner fasteners, are overdriven; and
(3) No more than 20% of fasteners in the field of the panel are overdriven.

C4.1.4.3.2 The *nominal shear strength* [*nominal resistance*] shall be multiplied by 0.75 where all the following criteria are met:

(1) In any of the four corners of an individual *wood structural panel*, none of the three fasteners closest to a panel's corner are overdriven;
(2) No more than 25% of fasteners along the perimeter, excluding the panel corner fasteners, are overdriven; and
(3) No more than 50% of fasteners in the field of the panel are overdriven.

User Note:
Figure C4.1.4-1 illustrates the corner, perimeter and field fasteners in structural wood panels.

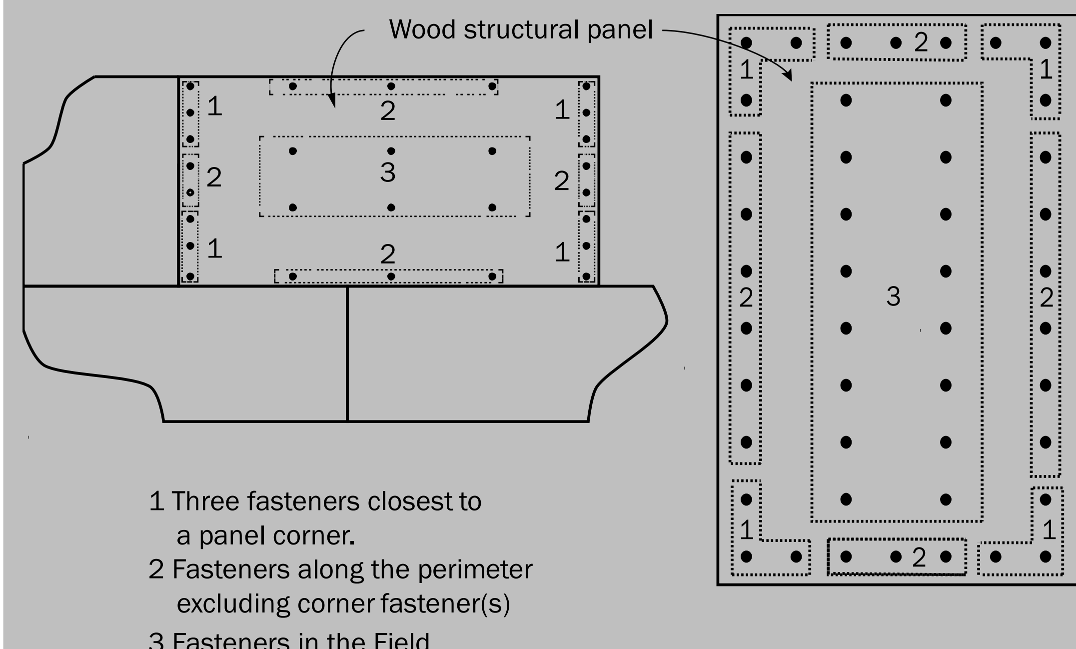

Figure C4.1.4-1 Fasteners in a Structural Wood Panel Diaphragm or Shear Wall

C4.2 Welded Connections

Welded areas not located within the building envelope or not shielded from direct contact with moisture from the ground or the exterior climate shall be protected with an *approved* treatment to retain the corrosion resistance of the welded area.

C5 Miscellaneous

Utilities and insulation shall be installed in accordance with the requirements of this section, as applicable.

C5.1 Utilities

C5.1.1 Holes

C5.1.1.1 Holes shall comply with the requirements specified in Section C2.1.

C5.1.1.2 Penetrations of floor, wall and ceiling/roof assemblies which are required to have a fire resistance rating shall be protected in accordance with the *approved construction documents*.

C5.1.2 Plumbing

All piping shall be provided with an isolative non-corrosive system to prevent galvanic action or abrasion between framing members and piping.

C5.1.3 Electrical

Wiring not enclosed in metal conduit shall be separated from the framing members by nonconductive, noncorrosive grommets or by other *approved* means.

C5.2 Insulation

C5.2.1 Mineral Fiber Insulation

Mineral fiber insulation (e.g., rock wool, glass fiber, etc.) for installation within cavities of framing members shall be full-width type insulation and shall be installed in accordance with the *applicable building code* and insulation manufacturer's requirements. Compression of the insulation is permitted to occur at the open side of the *C-shaped* framing member.

C5.2.2 Other Insulation

Other types of insulation (e.g., foams, loose fill, etc.) for installation within cavities of framing members shall be installed in accordance with the *applicable building code* and insulation manufacturer's requirements. The width of insulation shall be dimensionally compatible with the *cold-formed steel* framing.

D. QUALITY CONTROL AND QUALITY ASSURANCE

D1 General

D1.1 Scope and Limits of Applicability

Chapter D provides minimum requirements for *quality control*, *basic frame inspection* and *quality assurance* for material control, component manufacturing, and installation for *cold-formed steel light-frame construction* where required by the *applicable building code*. Minimum observation and *inspection* tasks deemed necessary to ensure quality *cold-formed steel light-frame construction* are specified.

D1.2 Responsibilities

Quality control as specified in this chapter shall be provided by the *component manufacturer* and *installer*, as applicable. *Basic frame inspection* as specified in this chapter shall be provided by the *authority having jurisdiction* where required by the *applicable building code*. *Quality assurance* as specified in this chapter shall be provided by the *quality assurance inspector* where required by the *authority having jurisdiction*, the *applicable building code*, the *owner*, or the *registered design professional*.

D2 Quality Control Programs

D2.1 The *component manufacturer* shall establish and maintain *quality control* procedures and perform *inspections* to ensure that the work is in accordance with this Standard and the *shop drawings*. Where the *component manufacturer's quality control* procedures are subject to the supervision of an *approved* third-party *quality control* agency in accordance with the *applicable building code*, submittal of *quality control* documents in accordance with Section D3.1 shall not be required.

> **User Note:**
> The exemption given in Section D2.1 to a *component manufacturer* subject to the supervision of an *approved* third-party *quality control* agency for the submission of *quality control* documents does not preclude the submission of submittals or documents in accordance with other sections of this Standard or Chapter I of AISI S202.

D2.2 The *installer* shall establish and maintain *quality control* procedures and perform *inspections* to ensure that the work is in accordance with this Standard and the *construction documents*.

D2.3 Product identification shall comply with the requirements of Section A5.5, and shall be monitored by the *component manufacturer's* and *installer's quality control inspectors*, as applicable.

D2.4 Where part of a *component assembly*, the *component manufacturer's quality control inspector* shall perform *inspections* of the following items, as applicable:

(a) *Cold-formed steel structural members* and *connectors* in accordance with Section D6.6.
(b) Shop welding in accordance with Section D6.7.
(c) Mechanical fastening in accordance with Section D6.8.

D2.5 The *installer's quality control inspector* shall perform *inspections* of the following, as applicable:

(a) *Cold-formed steel structural members* and *connectors* that are not part of a *component assembly* in accordance with Section D6.6.
(b) Field welding in accordance with Section D6.7.
(c) Mechanical fastening that is not part of a *component assembly* in accordance with Section D6.8.
(d) Installation of *cold-formed steel light-frame construction* in accordance with Section D6.9.
(e) Installation of *cold-formed steel lateral force-resisting systems* in accordance with Section D6.10.
(f) Damage to *cold-formed steel structural members* and *connectors*, including *component assemblies* damaged during or after delivery to the project.

D3 Quality Control Documents

D3.1 Documents to be Submitted

The *component manufacturer* and *installer* shall submit the following documents to the *registered design professional* and *contractor* for review prior to the installation of the *cold-formed steel light-frame construction*:

(a) *Shop drawings*, unless *shop drawings* have been furnished by others.
(b) *Installation drawings*, unless *installation drawings* have been furnished by others.
(c) Product data sheets, catalogue data or independent evaluation reports on *cold-formed steel structural members*, including material, corrosion protection, *base steel thickness*, and dimensions. Alternatively, material, corrosion protection, *base steel thickness*, and dimensions shall be shown on the *installation drawings*.
(d) Product data sheets, catalogue data or independent evaluation reports on *connectors* and mechanical fasteners.

D3.1.1 Additional Requirements for Lateral Force-Resisting Systems

D3.1.1.1 Component Assemblies

For *component assemblies*, the *component manufacturer* shall submit the following additional documents, as applicable, to the *registered design professional* and *contractor* for approval prior to the installation of the *cold-formed steel lateral force-resisting system* elements:

(a) Welding procedure specifications.
(b) Mechanical fastener installation procedures.
(c) Product data sheets, catalogue data or independent evaluation reports on *hold-downs*.

D3.1.1.2 Other Than Component Assemblies

For other than *component assemblies*, the *installer* shall submit the following additional documents, as applicable, to the *registered design professional* and *contractor* for approval prior to the installation of the *cold-formed steel lateral force-resisting system* elements:

(a) Welding procedure specifications.
(b) Mechanical fastener installation procedures.
(c) Product data sheets, catalogue data or independent evaluation reports on *hold-downs*.

D3.1.1.3 Exceptions to Sections D3.1.1.1 and D3.1.1.2

The additional requirements in Section D3.1.1.1 and Section D3.1.1.2 are not required for *cold-formed steel lateral force-resisting systems* where either of the following apply:

(a) The sheathing of the *shear wall* is gypsum board or *fiberboard*.

(b) The sheathing is wood *structural sheathing* or *steel sheet* sheathing on only one side of the *shear wall* or *diaphragm* assembly and the fastener spacing of the sheathing is more than 4 in. (102 mm) on center (o.c.).

D3.2 Available Documents

Unless required for submittal by the *registered design professional*, the following documents, as applicable and upon request, shall be made available by the *component manufacturer* or *installer*, as applicable, in electronic or printed form to the *registered design professional* and *contractor* for review prior to installation of the *cold-formed steel light-frame construction*:

(a) Manufacturer's installation instructions for *connectors*, *hold-downs* and mechanical fasteners.

(b) Manufacturer's product data sheets or catalog data for welding consumables, filler metals and fluxes that include the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements.

(c) Welding procedure specifications.

(d) Procedure qualification records for welding procedure specifications that are not prequalified in accordance with AWS D1.3, as applicable.

(e) Welding personnel performance qualification records and continuity records.

(f) *Component manufacturer's* and *installer's* written *quality control program(s)* that include material control procedures, *inspection* procedures, and nonconformance procedures.

(g) *Component manufacturer's* and *installer's quality control inspector* qualifications.

D4 Quality Assurance Agency Documents

The agency responsible for *quality assurance* shall submit the following documents to the *authority having jurisdiction*, *registered design professional* and *owner*, as applicable:

(a) *Quality assurance* agency's written practices for the monitoring and control of the agency's operations. The written practice shall include:

  (1) The agency's procedures for the selection and administration of *inspection* personnel, describing the training, experience and examination requirements for qualification and certification of *inspection* personnel.

  (2) The agency's *inspection* procedures, including general *inspection*, material controls, and visual welding *inspection*.

(b) Qualifications of management designated for the project.

(c) Qualification records for inspectors designated for the project.

D5 Inspection Personnel

D5.1 Quality Control Inspector

D5.1.1 A *quality control inspector* shall be designated as the person with overall responsibility for *quality control*. It is permitted to delegate specific quality control tasks to qualified personnel.

D5.1.2 *Quality control* welding *inspection* personnel shall be qualified in accordance with the *component manufacturer's* or *installer's quality control program*, as applicable, and in accordance with one of the following:

(a) Associate Welding Inspector (AWI) or higher as defined in AWS B5.1.

(b) Qualified by training or experience, or both, in *cold-formed steel light-frame construction* component manufacturing, installation, *inspection*, or testing and competent to perform *inspection* of the work.

D5.1.3 *Quality control* mechanical fastener *inspection* personnel shall be qualified in accordance with the *installer's quality control program* on the basis of training and experience in installation of similar fasteners and shall be competent to perform *inspection* of the work.

D5.2 Quality Assurance Inspector

D5.2.1 The *approved quality assurance inspector* shall be designated as the person with overall responsibility for *quality assurance*. The *approved quality assurance inspector* is permitted to delegate specific *quality assurance* tasks to qualified personnel.

D5.2.2 *Quality assurance* welding *inspection* personnel shall be qualified in accordance with the *quality assurance* agency's written practice and with either of the following:

(a) Welding Inspector (WI) or higher as defined in AWS B5.1, except Associate Welding Inspectors (AWI) shall be permitted to be used under the direct supervision of WIs or higher who are on the premises and available when weld *inspection* is being conducted; or

(b) Qualified by training or experience, or both, in *cold-formed steel light-frame construction* installation, *inspection* or testing and competent to perform *inspection* of the work.

D5.2.3 *Quality assurance* mechanical fastener *inspection* personnel shall be qualified in accordance with the *quality assurance* agency's written practice on the basis of training and experience in *inspection* of similar fasteners.

D6 Inspection Tasks

D6.1 General

Inspection tasks and documentation for *quality control*, *basic frame inspection* and *quality assurance* shall be in accordance with the tables in Sections D6.6, D6.7, D6.8, D6.9, and D6.10. These tables specify distinct *inspection* tasks as follows:

(a) *Observe*. Observe shall mean to perform *inspection* of these items on an intermittent basis. Operations that do not interfere with the ability to perform *inspection* of these items need not be delayed pending these *inspections*. Frequency of observations shall be adequate to confirm that the work has been performed in accordance with the applicable documents.

(b) *Perform*. Perform shall mean to execute these tasks prior to final acceptance for each item or element.

(c) *Document*. Within the listed tasks, document shall mean the individual performing the *inspection* tasks shall prepare reports or other written documentation indicating that the work has or has not been performed in accordance with the *construction documents*.

D6.2 Quality Control Inspection Tasks

D6.2.1 *Quality control inspection* tasks shall be performed by the *component manufacturer's* or *installer's quality control inspector*, as applicable, in accordance with Sections D6.6 through D6.10. Tasks in the tables in Sections D6.6 through D6.10 listed for QC shall be those *inspections* performed by the *quality control inspector* to ensure that the work is performed in accordance with the *construction documents*.

D6.2.2 *Quality control inspection* shall utilize the following, as applicable: *construction documents*, *installation drawings*, *shop drawings*, *plans*, *specifications,* and referenced standards.

D6.3 Basic Frame Inspection Tasks

D6.3.1 *Basic frame inspection* of the *component assemblies* shall be made at the project site. The *contractor* shall schedule this work with the *authority having jurisdiction* and the *installer* to minimize interruptions to the work of the *installer*.

D6.3.2 *Basic frame inspection* of the *cold-formed steel light-frame construction* shall be made at the project site. The *contractor* shall schedule this work with the *authority having jurisdiction* and the *installer* to minimize interruptions to the work of the *installer*.

D6.3.3 The *authority having jurisdiction* shall review the materials test reports and certifications listed in Section D3.2 for compliance with the *construction documents*.

D6.3.4 *Basic frame inspection* tasks shall be performed by the *authority having jurisdiction* in accordance with Sections D6.6 through D6.10. Tasks in the tables in Sections D6.6 through D6.10 listed for BFI shall be those *inspections* performed by the *authority having jurisdiction* to ensure that the work is performed in accordance with the *construction documents*.

D6.3.5 Concurrent with the submittal of reports to the *registered design professional* and *owner*, as applicable, the *authority having jurisdiction* shall submit to the *contractor* and the *installer* lists of nonconforming items.

D6.4 Quality Assurance Inspection Tasks

D6.4.1 *Quality assurance inspection* of the *component assemblies* shall be made at the *component manufacturer's* plant. The *quality assurance inspector* shall schedule this work to minimize interruptions to the work of the *component manufacturer*.

D6.4.2 *Quality assurance inspection* of the *cold-formed steel light-frame construction* shall be made at the project site. The *contractor* shall schedule this work with the *quality assurance inspector* and the *installer* to minimize interruptions to the work of the *installer*.

D6.4.3 The *quality assurance inspector* shall review the materials test reports and certifications listed in Section D3.2 for compliance with the *construction documents*.

D6.4.4 *Quality assurance* tasks shall be performed by the *quality assurance inspector*, in accordance with Sections D6.6 through D6.10. Tasks in the tables in Sections D6.6 through D6.10 listed for QA shall be those *inspections* performed by the *quality assurance inspector* to ensure that the work is performed in accordance with the *construction documents*.

D6.4.5 Concurrent with the submittal of reports to the *authority having jurisdiction*, *registered design professional* and *owner*, as applicable, the *quality assurance inspector* shall submit to the *contractor* and the *installer* lists of nonconforming items.

D6.5 Coordinated Inspection

D6.5.1 *Quality assurance inspection* of the *component assemblies* shall be made at the *component manufacturer's* plant. The *quality assurance inspector* shall schedule this work to minimize interruptions to the work of the *component manufacturer*. Where the *component manufacturer* has been *approved* to perform such work without *inspection* in accordance with the *applicable building code*, *quality assurance inspection* shall not be required. The *component manufacturer* shall still perform and document all of the required in-plant *quality assurance inspections*.

D6.5.2 Where *quality assurance* tasks are performed only by the *quality control inspector*, each *inspection* shall be documented in a report and the *quality assurance inspector* shall periodically review the work of the *quality control inspector* at an interval acceptable to the *owner*, *registered design professional*, and *authority having jurisdiction*.

D6.6 Material Verification

D6.6.1 The *component manufacturer's quality control inspector* shall perform *inspections* of the *cold-formed steel structural members* and *connectors* used in *component assemblies* to verify compliance with the details shown on the *shop drawings*.

D6.6.2 The *installer's quality control inspector* shall perform *inspections* of the *cold-formed steel structural members* and *connectors* that are not part of a *component assembly* to verify compliance with the details shown on the *installation drawings*.

D6.6.3 The *authority having jurisdiction* and *quality assurance inspector*, as applicable, shall perform verifications and *inspections*, as applicable, to verify compliance with the *construction documents* and this Standard.

D6.6.4 *Inspection* tasks shall be in accordance with Tables D6.6-1 and D6.6-2.

Table D6.6-1
Material Verification Tasks
Prior to Assembly or Installation

	Task	QC	BFI	QA
A	Verify compliance of *cold-formed steel structural members*: - Product identification (Section A5.5)	Perform	Not Required	Not Required
B	Verify compliance of *connectors*	Perform	Not Required	Not Required
C	Document acceptance or rejection of *cold-formed steel structural members* and *connectors*	Not Required[1]	Not Required	Not Required

[1] Documentation tasks for *quality control* are as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

Table D6.6-2
Material Verification Tasks
After Assembly or Installation

	Task	QC	BFI	QA
A	Verify compliance of *cold-formed steel structural members*: - Product identification (Section A5.5)	Perform	Observe	Not Required
B	Verify compliance of *connectors*	Perform	Observe	Not Required
C	Document acceptance or rejection of *cold-formed steel structural members* and *connectors*	Not Required[1]	Perform	Not Required

[1] Documentation tasks for *quality control* are as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

D6.7 Inspection of Welding

D6.7.1 Observation of welding operations and visual *inspection* of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the *construction documents* and AWS D1.3.

D6.7.2 Welding *inspection* tasks shall be in accordance with Tables D6.7-1, D6.7-2 and D6.7-3.

Table D6.7-1
Inspection or Execution Tasks
Prior to Welding

	Task	QC	BFI	QA
A	Welding procedure specifications available	Observe	Not Required	Not Required
B	Manufacturer certifications for welding consumables available	Observe	Not Required	Not Required
C	Material identification (type/grade)	Observe	Not Required	Not Required
D	Check welding equipment	Observe	Not Required	Not Required

Table D6.7-2
Inspection or Execution Tasks
During Welding

	Task	QC	BFI	QA
A	Use of qualified welders	Observe	Not Required	Observe
B	Control and handling of welding consumables	Observe	Not Required	Not Required
C	Environmental conditions (wind speed, moisture, temperature)	Observe	Not Required	Not Required
D	Welding procedure specifications followed	Observe	Not Required	Not Required

Table D6.7-3
Inspection or Execution Tasks
After Welding

	Task	QC	BFI	QA
A	Verify compliance of welds	Perform	Observe	Perform
B	Welds meet visual acceptance criteria	Perform	Observe	Perform
C	Verify repair activities	Perform	Observe	Perform
D	Document acceptance or rejection of welded connections	Not Required[1]	Perform	Perform

[1] Documentation tasks for *quality control* are as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

D6.8 Inspection of Mechanical Fastening

D6.8.1 Observation of mechanical fastening operations and visual *inspection* of in-process and completed connections shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the *construction documents* and manufacturer's installation instructions.

D6.8.2 Mechanical fastening *inspection* tasks shall be in accordance with Tables D6.8-1, D6.8-2 and D6.8-3.

Table D6.8-1
Inspection or Execution Tasks
Prior to Mechanical Fastening

	Task	QC	BFI	QA
A	Mechanical fastener manufacturer installation instructions available for mechanical fasteners	Observe	Not Required	Not Required
B	Proper tools available for mechanical fastener installation	Observe	Not Required	Not Required
C	Proper storage for mechanical fasteners	Observe	Not Required	Not Required

Table D6.8-2
Inspection or Execution Tasks
During Mechanical Fastening

	Task	QC	BFI	QA
A	Mechanical fasteners are positioned as required	Observe	Not Required	Not Required
B	Mechanical fasteners are installed in accordance with manufacturer's instructions	Observe	Not Required	Not Required

Table D6.8-3
Inspection or Execution Tasks
After Mechanical Fastening

	Task	QC	BFI	QA
A	Verify compliance of mechanical fasteners	Perform	Observe	Not Required
B	Repair activities	Perform	Observe	Not Required
C	Document acceptance or rejection of mechanically fastened connections	Not Required [1]	Perform	Not Required

[1] Documentation tasks for *quality control* are as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

D6.9 Inspection of Cold-Formed Steel Light-Frame Construction

D6.9.1 The *component manufacturer's quality control inspector* shall perform *inspections* of the *component assemblies* to verify compliance with the details shown on the *shop drawings*.

D6.9.2 The *installer's quality control inspector* shall perform *inspections* of the field-installed *cold-formed steel structural members*, *component assemblies* and *connectors* to verify compliance with the details shown on the *installation drawings*.

D6.9.3 The *authority having jurisdiction* and *quality assurance inspector*, as applicable, shall perform verifications and *inspections*, as applicable, to verify compliance with the *construction documents*.

D6.9.4 *Inspection* tasks shall be in accordance with Table D6.9-1.

Table D6.9-1
Inspection or Execution Tasks
After Installation of Cold-Formed Steel Light-Frame Construction

	Task	QC	BFI	QA
A	Verify compliance of *cold-formed steel light-frame construction*	Perform	Observe	Not Required
B	Document acceptance or rejection of *cold-formed steel light-frame construction*	Not Required [1]	Perform	Not Required

[1] Documentation tasks for *quality control* are as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

D6.10 Additional Requirements for Lateral Force-Resisting Systems

Where special inspection is required for wind resistance or seismic resistance by the *applicable building code*, additional *inspection* tasks for *cold-formed steel lateral force-resisting systems* shall be in accordance with Tables D6.10-1 through D6.10-5.

Exception: The requirements in Section D6.10 are not required for *cold-formed steel lateral force-resisting systems* where either of the following apply:

(a) The sheathing of the *shear wall* is gypsum board or *fiberboard*.

(b) The sheathing is wood *structural sheathing* or *steel sheet* sheathing on only one side of the *shear wall* or *diaphragm* assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

User Note:
Special inspection for high wind resistance and high seismic resistance is addressed in Chapter 17 of the IBC.

D6.10.1 Fit-Up of Welds

In Table D6.10-2, following performance of the *inspection* task for 10 welds made by a given welder, with the welder demonstrating understanding of requirements and possession of skills, the Perform designation of this task shall be reduced to Observe.

Table D6.10-1
Additional Inspection or Execution Tasks
Prior to Installation of Cold-Formed Steel Lateral Force-Resisting Systems

	Task	QC	BFI	QA
A	Verify compliance of *shear wall* and *diaphragm* sheathing, diagonal *strap bracing*, and *hold-downs*, as applicable	Perform	Not Required	Not Required
B	Document acceptance or rejection of *shear wall* and *diaphragm* sheathing, diagonal *strap bracing*, and *hold-downs*, as applicable	Not Required[1]	Not Required	Not Required

[1] Documentation tasks for *quality control* should be as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

Table D6.10-2
Additional Inspection or Execution Tasks
Prior to Welding of Cold-Formed Steel Lateral Force-Resisting Systems

	Task	QC	BFI	QA
A	Welder identification system[1]	Observe	Not Required	Not Required
B	Fit-up of welds (alignment, gaps, condition of steel surfaces)	Perform/ Observe[2]	Not Required	Not Required

[1] A system maintained by the *component manufacturer* or *installer*, as applicable, by which a welder who has welded a joint or member can be identified.
[2] See Section D6.10.1.

Table D6.10-3
Additional Inspection or Execution Tasks
Prior to Mechanical Fastening of Cold-Formed Steel Lateral Force-Resisting Systems

	Task	QC	BFI	QA
A	Proper fasteners selected	Observe	Not Required	Not Required
B	Proper installation procedure selected	Observe	Not Required	Not Required
C	Connecting elements meet applicable requirements	Observe	Not Required	Not Required

Table D6.10-4
Additional Inspection or Execution Tasks
During Mechanical Fastening of Cold-Formed Steel Lateral Force-Resisting Systems

	Task	QC	BFI	QA
A	For screw connections, joint brought tight (e.g., clamped) to avoid gaps between plies	Observe	Not Required	Observe
B	For screw connections, tool adjusted to avoid stripped and overdriven fasteners	Observe	Not Required	Observe
C	For post-installed connections to concrete, installation in accordance with manufacturer's instructions	Perform	Not Required	Perform

Table D6.10-5
Additional Inspection or Execution Tasks
After Installation of Cold-Formed Steel Lateral Force-Resisting Systems

	Task	QC	BFI	QA
A	Verify compliance of *cold-formed steel lateral force-resisting system* installation: screw attachment, bolting, anchoring and other fastening of elements of the lateral force-resisting system, including shear walls, strap braced walls, braces, diaphragms, collectors (drag struts) and hold-down systems.	Perform	Not Required	Observe
B	Document acceptance or rejection of installation of *cold-formed steel lateral force-resisting system*	Not Required[1]	Not Required	Perform

[1] Documentation tasks for *quality control* are as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

D7 Nonconforming Material and Workmanship

D7.1 Additional Inspections

In the event that observations determine that the materials and/or workmanship are not in conformance with the applicable documents, additional *inspections*, as specified by the *quality control program* or *registered design professional*, shall be performed to determine the extent of non-conformance.

D7.2 Rejection of Material

D7.2.1 Identification and rejection of materials and workmanship not in conformance with the *construction documents* is permitted at any time during progress of or following the completion of the work. However, this provision shall not relieve the *owner* or the inspector of the obligation for timely, in-sequence *inspections*.

D7.2.2 Nonconforming material and workmanship shall be brought to the immediate attention of the *contractor* and the *installer*.

D7.2.3 Nonconforming material or workmanship shall be brought into conformance, or made suitable for its intended purpose, as determined by the *registered design professional*.

E. TRUSSES

E1 General

Cold-formed steel trusses shall be designed in accordance with the requirements of this chapter, as applicable.

E1.1 Scope and Limits of Applicability

E1.1.1 Chapter E shall apply to design, manufacturing, quality criteria, installation, and testing as they relate to the design of *cold-formed steel trusses*.

E1.1.2 The responsibilities specified in Section E2 are not intended to preclude alternate provisions as agreed upon by the parties involved.

E2 Truss Responsibilities

Truss responsibilities shall be in accordance with Section I1 of AISI S202.

E3 Loading

[Reserved]

E4 Truss Design

Except as modified or supplemented in this Standard, strength determinations shall be in accordance with AISI S100 [CSA S136].

E4.1 Materials

Sheet steel materials utilized in steel *truss* construction shall comply with ASTM A1003/A1003M Type H or ASTM A653/A653M Type SS, HSLAS, or HSLAS-F. *Cold-formed steel* welded tubing utilized in steel *truss* construction shall comply with ASTM A500.

E4.2 Corrosion Protection

Truss members, including *gusset plates*, shall have corrosion protection as required in accordance with Section A4.

E4.3 Analysis

In lieu of a *rational engineering analysis* to define joint flexibility, the following analysis modeling assumptions shall be used:

(1) *Chord members* are continuous, except at the *heel*, *pitch breaks*, and *chord splices* where members are assumed to have pinned *connections*.

(2) *Web members* are assumed to have pinned *connections* at each end.

Use of a specific joint stiffness other than the complete rotational freedom of a pin for a *connection* is permitted if the *connection* is designed for the forces resulting from a structural analysis with this specific joint stiffness.

E4.4 Member Design

E4.4.1 Properties of Sections

For *C-shapes* and other simple cross-section geometries, the properties of sections shall be determined in accordance with conventional methods of structural design *rational engineering analysis*. Properties shall be based on full cross-section properties, except where use of a reduced cross-section or effective design width is required by AISI S100 [CSA S136]. For other cross-section geometries, properties shall be based on tests in accordance with Chapter F.

E4.4.2 Compression Chord Members

The compression *chord member* shall be evaluated for axial *load* alone using Chapter E of AISI S100 [CSA S136], bending in accordance with Chapter F of AISI S100 [CSA S136], and combined axial *load* and bending in accordance with Section H1.2 of AISI S100 [CSA S136].

E4.4.2.1 For axial *load strength* [*resistance*] determination, the effective length, KL, shall be determined by *rational engineering analysis*, testing, or the following design assumptions as applicable:

(a) For *C-shape chord members* with the x-axis as the axis of symmetry: L_x shall be equal to the distance between *panel points*, and C_m shall be taken as 0.85, unless an analysis is performed to justify another value. Where the *chord member* is continuous over at least one intermediate *panel point* and where *structural sheathing* or *profiled steel panels* are directly attached to the *chord member*, K_x shall be taken as 0.75. Otherwise, K_x shall be taken as unity. As an alternative, L_x shall be the distance between points of contraflexure with C_m and K_x taken as unity. Where *structural sheathing* or *profiled steel panels* are directly attached to the *chord member*, L_y shall be equal to the distance between sheathing connectors and K_y shall be taken as 0.75. Where purlins are attached to the *chord member*, L_y shall be the distance between purlins with K_y equal to unity. L_t shall be equal to the distance between *panel points*. Where the *chord member* is continuous over at least one intermediate *panel point* between the *heel* and *pitch break* and where sheathing is directly attached to the *chord member*, K_t shall be taken as 0.75. Otherwise, K_t shall be taken as unity. As an alternate, L_t shall be the distance between points of contraflexure with K_t taken as unity. The variables are defined as follow:

C_m = End moment coefficient in interaction formula
K_t = Effective length factor for torsion
K_x = Effective length factor for buckling about x-axis
K_y = Effective length factor for buckling about y-axis
L_t = Unbraced length of compression member for torsion
L_x = Unbraced length of compression member for bending about x-axis
L_y = Unbraced length of compression member for bending about y-axis

(b) For *hat-shapes* with the x-axis as the axis of symmetry: Where *structural sheathing* or *profiled steel panels* are directly attached to the *chord member*, L_x shall be equal to the distance between sheathing connectors and K_x shall be taken as 0.75. Where purlins are attached to the *chord member*, L_x shall be the distance between purlins with K_x equal to unity. L_y shall be equal to the distance between *panel points*, and C_m shall be taken as 0.85, unless an analysis is performed to justify another value. Where the *chord member* is continuous over at least one intermediate *panel point* and where *structural sheathing* or *profiled steel panels* are directly attached to the *chord member*, K_y shall be taken as 0.75. Otherwise, K_y shall be taken as unity. As an alternative, L_y shall be the distance between points of contraflexure with C_m and K_y taken as unity. L_t shall be equal to the distance between sheathing connectors or purlin spacing. Where the *chord member* is continuous over at least one intermediate *panel point* between the *heel* and *pitch break* and where sheathing is directly attached to the *chord member*, K_t shall be taken as 0.75. Otherwise, K_t shall be taken as unity. As an alternate, L_t shall be the distance between the points of contraflexure with K_t taken as unity.

(c) For *Z-shapes* with the x-axis as out-of-the-plane of the *truss*: L_x shall be equal to the distance between *panel points*, and C_m shall be taken as 0.85, unless an analysis is performed to justify another value. Where the *chord member* is continuous over at least one intermediate *panel point* and where *structural sheathing* or *profiled steel panels* are directly attached to the *chord member*, K_x shall be taken as 0.75. Otherwise, K_x shall be taken as unity. As an alternative, L_x shall be the distance between points of contraflexure with C_m and K_x taken as unity. Where *structural sheathing* or *profiled steel panels* are directly attached to the *chord member*, L_y shall be equal to the distance between sheathing connectors and K_y shall be taken as 0.75. Where purlins are attached to the *chord member*, L_y shall be the distance between purlins with K_y equal to unity. Where the *chord member* depth is less than 6 inches, L_t shall be equal to the distance between sheathing connectors or purlin spacing. For *Z-shapes* where the *chord member* depth is greater than or equal to 6 inches, L_t shall be equal to the distance between *panel points*. Where the *chord member* is continuous over at least one intermediate *panel point* between the *heel* and *pitch break* and where sheathing is directly attached to the *chord member*, K_t shall be taken as 0.75. Otherwise, K_t shall be taken as unity. As an alternative, L_t shall be equal to the distance between points of contraflexure with K_t taken as unity.

E4.4.2.2 For bending strength [*resistance*] determination, the effective length, KL, shall be determined by *rational engineering analysis*, testing, or the following design assumptions as applicable:

(a) Where *structural sheathing* or *profiled steel panels* are directly attached to the compression *flange*, the *available strength* [*factored resistance*] is the lesser of the strength determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136].

(b) Where purlins are directly attached to the compression *flange* between *panel points*, the *available strength* [*factored resistance*] is determined in accordance with Chapter F of AISI S100 [CSA S136] with KL_y and KL_t for *C-shapes* and *Z-shapes* determined in accordance with E4.4.2.1 and KL_x and KL_t for *hat-shapes* taken as the distance between purlins.

(c) Where the compression *flange* is laterally unbraced, the *available* flexural *strength* [*factored resistance*] is determined in accordance with Chapter F of AISI S100 [CSA S136]. For continuous span *chord members*, M_n in the region of the *panel point* is determined with KL_y and KL_t for *C-shapes* and *Z-shapes* determined in accordance with E4.4.2.1 and KL_x and KL_t for *hat-shapes* taken as the distance between the *panel point* and the point of contraflexure, and C_b is taken as unity. For simple and continuous span *chord members*, M_n in the mid-span region is determined with the effective length taken as the distance between *panel points* and C_b shall be computed in accordance with Section F2.1.1 of AISI S100 [CSA S136]. The variables are defined as follows:

C_b = Bending coefficient dependent on moment gradient
F_c = Critical buckling stress
F_y = *Yield strength* used for design
K_t = Effective length factor for torsion
K_x = Effective length factor for buckling about x-axis
K_y = Effective length factor for buckling about y-axis
L_t = Unbraced length of compression member for torsion
L_x = Unbraced length of compression member for bending about x-axis
L_y = Unbraced length of compression member for bending about y-axis

E4.4.2.3 When a *C-shaped* section compression *chord member* is subject to concentrated *load* at a *panel point*, the interaction of axial compression, bending and web crippling shall be determined in accordance with the following:

For *ASD*:

$$\frac{P}{P_{no}} + \frac{M}{M_{nxo}} + \frac{R}{R_n} \leq \frac{1.49}{\Omega} \qquad (Eq.\ E4.4.2.3\text{-}1)$$

where

P = *Required* compressive axial *strength*
M_x = *Required* flexural *strength*
R = *Required* concentrated load *strength*
P_{no} = *Nominal* axial *strength* determined in accordance with Section E3 of AISI S100 [CSA S136] at $F_n = F_y$ or $P_{ne} = P_y$
M_{nxo}= *Nominal* flexural *strength* determined in accordance with Section F3 of AISI S100 [CSA S136] at $F_n = F_y$ or $M_{ne} = M_y$
R_n = *Nominal* interior one-*flange* web crippling *strength*
Ω = 1.95

For *LRFD* and *LSD*:

$$\frac{\overline{P}}{P_{no}} + \frac{\overline{M}_x}{M_{nxo}} + \frac{\overline{R}}{R_n} \leq 1.49\phi \qquad (Eq.\ E4.4.2.3\text{-}2)$$

where

$\overline{P}$ = *Required* compressive axial *strength* [axial force of *factored loads*]
$\overline{M}_x$ = *Required* flexural *strength* [moment of *factored loads*]
$\overline{R}$ = *Required* concentrated load *strength* [factored concentrated *load*]
P_{no} = *Nominal* axial *strength* [*resistance*] determined in accordance with Section E3 of AISI S100 [CSA S136] at $F_n = F_y$ or $P_{ne} = P_y$
M_{nxo}= *Nominal* flexural *strength* [*resistance*] determined in accordance with Section F3 of AISI S100 [CSA S136] at $F_n = F_y$ or $M_{ne} = M_y$
R_n = *Nominal* interior one-*flange* loading *web* crippling *strength* [*resistance*]
ϕ = 0.85 for *LRFD*
= 0.80 for *LSD*

E4.4.3 Tension Chord Members

The tension *chord member* shall be evaluated for axial *load* in accordance with Chapter D of AISI S100 [CSA S136], bending in accordance with Chapter F of AISI S100 [CSA S136], and combined axial *load* and bending in accordance with Section H1.1 of AISI S100 [CSA S136]. The axial *load* is permitted to be taken as acting through the centroid of the section.

E4.4.4 Compression Web Members

Compression *web members* shall be evaluated for axial *load* alone using Chapter E of AISI S100 [CSA S136] and combined axial *load* and bending using Section H1.2 of AISI S100 [CSA S136], and the requirements of this section, as applicable.

E4.4.4.1 For a *C-shaped* compression *web member* that is attached at each end through its *web* element back-to-back with the *web* of a *C-shaped chord member* and is not subjected to applied *loads* between its ends, the interaction of axial compression and out-of-plane bending shall be determined in accordance with the following interaction equation:

$$\frac{R\overline{P}}{P_a} + \frac{C_{my} R \overline{P} e}{M_{ay} \alpha_y} \le 1.0 \qquad (Eq.\ E4.4.4\text{-}1)$$

where

$$R = -\left(\frac{L/r}{173}\right)^2 + \frac{L/r}{88} - 0.22 \ge 0.6 \qquad (Eq.\ E4.4.4\text{-}2)$$

L = Unbraced length of the compression *web member*

r = Radius of gyration of the full section about the minor axis

$\overline{P}$ = *Required* compressive *strength* [compressive axial force due to *factored loads*] determined in accordance with *ASD*, *LRFD* or *LSD* load combinations

e = Eccentricity of compression force with respect to the centroid of the full section of the *web member*

P_a = *Available* axial *strength* [*factored resistance*] based on Sections E2 and E3 of AISI S100 [CSA S136]. Only flexural and local buckling need to be considered

M_{ay} = *Available* flexural *strength* [*factored resistance*] based on Sections F2 and F3 of AISI S100 [CSA S136]. Only global and local buckling need to be considered

C_m = End moment coefficient in interaction formula in accordance with Section E4.4.2

$$\alpha_y = 1 - \frac{c\overline{P}}{P_{Ey}} \qquad (Eq.\ E4.4.4\text{-}3)$$

c = 1.80 for *ASD*

= 1.0 for *LRFD* or *LSD*

When computing the *available strength* [*factored resistance*], the effective lengths, K_xL_x, K_yL_y and K_tL_t, shall be taken as the distance between the centers of the member's end *connection* patterns.

Alternatively, it is permitted to determine the *required strengths* [forces and moments due to *factored loads*] of $\overline{P}$ and $\overline{M}$ in accordance with Section C1 of AISI S100 [CSA

S136] with eccentricity, e, enclosed in the analysis, and perform the interaction check in accordance with Section H2 of AISI S100 [CSA S136], where the distortional buckling are permitted to be ignored in determining member *available strengths* [*factored resistance*] of P_a and M_a.

Variables $\overline{P}$ and $\overline{M}$, P_a and M_a are defined in Section H2 of AISI S100 [CSA S136].

E4.4.4.2 For other than *C-shape* compression *web members* that are concentrically loaded, the axial compression *load* is permitted to be taken as acting through the centroid of the section.

E4.4.4.3 For other than *C-shape* compression *web members* that are not concentrically loaded, proper regard for eccentricity shall be considered.

E4.4.5 Tension Web Members

Tension *web members* shall be evaluated for axial *load* in accordance with Chapter D of AISI S100 [CSA S136]. For tension *web members*, which are symmetrically loaded, the axial tension *load* is permitted to be taken as acting through the centroid of the section. For other tension members that are not symmetrically loaded, proper regard for eccentricity shall be considered.

E4.4.6 Eccentricity in Joints

E4.4.6.1 A *rational engineering analysis* using multiple nodes or an analysis using single node that includes proper regard for the effects of eccentricity shall be performed.

E4.4.6.2 *Chord member* shear and moments in joints shall include the following considerations:

(a) Where the *web member* lap length is greater than or equal to 75% of the *chord member* depth, the *chord member* shall be investigated for combined bending and shear in accordance with Equation H2-2 of AISI S100 [CSA S136]. For *C-shaped* section *trusses* where screws are used as the connector, a minimum of four screws shall be used in the *web member* to *chord member connection* and the screws shall be uniformly distributed in the lapped area.

(b) Where the *web member* lap length is less than 75% of the *chord member* depth, the *chord* member shall be investigated for combined bending and shear in accordance with Equation H2-1 of AISI S100 [CSA S136].

E4.4.6.3 Along the length of the *chord member*, at the mid-point between the intersecting *web members* at a joint, shear shall be evaluated by Section G2 of AISI S100 [CSA S136]. The shear buckling coefficient shall be based on either Equation G2.3-3 or G2.3-4 with "a" taken as the smaller of the distance between the fastener groups, or center-to-center of the *web members.*

E4.5 Gusset Plate Design

E4.5.1 The *nominal* axial compressive *strength* [*resistance*], P_n, of thin, flat *gusset plates* shall be calculated as follows:

$$P_n = R_g b t F_y \qquad (Eq.\ E4.5\text{-}1)$$

where

$$R_g = \left(0.47\frac{W_{min}}{L_{eff}} + 0.3\right) \quad \text{where } \frac{W_{min}}{L_{eff}} \le 1.5 \qquad (Eq.\ E4.5\text{-}2)$$

=1.0 where $\frac{W_{min}}{L_{eff}} > 1.5$

b = Effective width determined in accordance with Appendix 1 Section 1.1 of AISI S100 [CSA S136] with $f=F_y$, k=4 and $w=W_{min}$

F_y = Specified minimum *yield strength*

t = *Design thickness* of *gusset plate*

Ω_c = 2.50 for *ASD*

ϕ_c = 0.60 for *LRFD*

= 0.50 for *LSD*

W_{min} shall be taken as the lesser of the actual *gusset plate* width or Whitmore section, which shall be determined using a spread-out angle of 30° along both sides of the *connection*, beginning at the first row of fasteners in the *connection*. L_{eff} shall be taken as the average length between the last rows of fasteners of adjacent *truss* members in the *connection*.

Equation E4.5-1 shall be valid for the range of parameters listed in Table E4.5-1.

Table E4.5-1
Parameters for Equation E4.5-1

Parameter	Minimum	Maximum
Gusset Plate:		
Design Thickness	0.0566 inch (1.438 mm)	0.1017 inch (2.583 mm)
Design Yield Strength	33 ksi (228 MPa)	50 ksi (345 MPa)
W_{min} / L_{eff} Ratio	0.8	6.0
Chord Member-to-Gusset Plate Fastener Pattern	Two rows with two fasteners per row	n/a

E4.5.2 The *nominal* axial tensile *strength* [*resistance*] of thin, flat *gusset plates* shall be calculated in accordance with the requirements of Chapter D of AISI S100 [CSA S136].

E4.6 Connection Design

E4.6.1 Fastening Methods

Fastening systems shall be specified by the *truss designer*. Screw, bolt, and weld *connections* shall be designed in accordance with AISI S100 [CSA S136]. For *connections* using other fastener types, design values shall be determined by testing in accordance with Section K2.1 of AISI S100 [CSA S136].

Other fastening methods shall be in accordance with the manufacturer's requirements.

E4.6.2 Coped Connections for C-Shaped Sections

E4.6.2.1 Coping is permitted at *pitch break* and *heel connections* in accordance with the *truss* design and the following, as applicable:

(a) At a coped *heel connection* with a coped *flange* and a *bearing stiffener* having a moment of inertia (I_{min}) greater than or equal to 0.161 in.^{4} (67,000 mm^4), the *available* shear *strength* [*factored resistance*] shall be calculated in accordance with Section G2 of AISI S100 [CSA S136] and reduced by the following factor, R:

$$R = 0.976 - \frac{0.556c}{h} - \frac{0.532d_c}{h} \leq 1.0 \quad (Eq.\ E4.6.2\text{-}1)$$

where

c = Length of cope

d_c = Depth of cope

h = Flat width of *web* of section being coped

I_{min} = Moment of inertia determined with respect to an axis parallel to the *web* of the *chord member*

t = *Design thickness* of section being coped

(b) At a coped *heel connection* with a coped *flange* where a *bearing stiffener* having a moment of inertia (I_{min}) less than 0.161 $in.^4$ (67,000 mm^4), the strength at the *heel* is governed by *web* crippling determined in accordance with Section G5 of AISI S100 [CSA S136] and reduced by the following factor, R:

$$R = 1.036 - \frac{0.668c}{h} - \frac{0.0505d_c}{h} \leq 1.0 \quad (Eq.\ E4.6.2\text{-}2)$$

E4.6.2.2 Equations E4.6.2-1 and E4.6.2-2 shall be applicable within the following limitations:

$h/t \leq 200$,

$0.10 < c/h < 1.0$, and

$0.10 < d_c/h < 0.4$

E4.7 Serviceability

Serviceability requirements, as specified in AISI S100 [CSA S136], shall be determined by the *building designer* or *applicable building code*. When computing *truss* deflections, it is permitted to use the full cross-sectional area of the *truss* members.

E5 Quality Criteria for Steel Trusses

Section E5 applies to the manufacture of *cold-formed steel trusses*.

E5.1 Manufacturing Quality Criteria

The *truss manufacturer* shall manufacture the *trusses* in accordance with the final *truss design drawings*, using the quality criteria required by the manufacturer's quality control program unless more stringent quality criteria are required by the owner in writing or through the *construction documents*.

E5.2 Member Identification

Truss chord members and *web members* shall be identified in accordance with the product identification requirements for framing members defined in Section A5.5.

E5.3 Assembly

E5.3.1 *Trusses* shall have steel members that are accurately cut, in accordance with the *truss* design, so that the assembled *truss* is made by close-fitting steel members.

E5.3.2 The maximum gap between *web members* shall not exceed 1/2 inch (12.7 mm) unless specified or accepted by the *truss design engineer* or *truss designer*.

E5.3.3 The location of *chord members*, *web members*, and joints shall be as specified in the *truss* design.

E5.3.4 *Truss* dimensions which vary from the *truss* design shall not exceed the tolerances shown in Table E5.3-1. Inaccuracies exceeding these allowable tolerances shall be acceptable upon approval and follow-up documentation by the *truss design engineer* or *truss designer*.

E5.3.5 Any shop modifications or repairs shall be documented by the *truss design engineer* or *truss designer*.

Table E5.3-1
Manufacturing Tolerances for Finished Truss Units

Length[1]	Variance from Design Dimensions
Up to 30 feet (9.14 m)	½ inch (12.7 mm)
Over 30 feet (9.14 m)	¾ inch (19.1 mm)
Height[2]	**Variance from Design Dimensions**
Up to 5 feet (1.52 m)	¼ inch (6.4 mm)
Over 5 feet (1.52 m)	½ inch (12.7 mm)

1 Length, for manufacturing tolerance purposes, is the overall length of the *truss* unit, excluding overhangs and extensions.

2 Height, for manufacturing tolerance purpose, is the overall height of the *truss* unit measured from the top of the top *chord member* to the bottom of the bottom *chord member* at the highest point of the *truss*, excluding projections above the top *chord member* and below the bottom *chord member*, overhangs, and extensions.

E6 Truss Installation

Section E6 applies to installation of *cold-formed steel trusses*.

E6.1 Installation Tolerances

E6.1.1 Straightness

Trusses shall not be installed with an overall bow or bow in any *chord member* or panel which exceeds the lesser of L/200 or 2 inches (50.8 mm), where L is the length of the *truss*, *chord member*, or panel in inches.

E6.1.2 Plumbness

Trusses shall not be installed with a variation from plumb (vertical tolerance) at any point along the length of the *truss* from top to bottom which exceeds 1/50 of the depth of the *truss* at that point or 2 inches (50.8 mm), whichever is less, unless *trusses* are specifically designed to be installed out of plumb.

E6.1.3 Top Chord Bearing Trusses

For top *chord* bearing *trusses*, a maximum gap tolerance between the inside of the bearing and the first diagonal or vertical *web member* shall be specified in the design.

E7 Test-Based Design

Tests, when required as defined below, shall be conducted under the supervision of a *registered design professional* in accordance with the following:

(a) For *cold-formed steel truss* components (*chord members* and *web members*) for which the *nominal strength* [*resistance*] cannot be calculated in accordance with this Standard or AISI S100 [CSA

S136], performance tests shall be performed in accordance with Chapter F of this Standard.

(b) For *cold-formed steel truss connections* for which the *nominal strength* [*resistance*] cannot be determined in accordance with this Standard or its reference documents, performance tests shall be performed in accordance with AISI S905.

(c) For *cold-formed steel trusses* for which the *nominal strength* [*resistance*] can be determined in accordance with this Standard and its reference documents or determined on the basis of component performance tests in accordance with E7(a), and when it must be demonstrated that the strength [*resistance*] is not less than the *nominal strength* [*resistance*] specified in this Standard or its reference documents, confirmatory tests shall be performed in accordance with AISI S921.

(d) For *cold-formed steel trusses* for which the *nominal strength* [*resistance*] cannot be determined in accordance with this Standard and its reference documents or determined on the basis of component performance tests in accordance with E7(a), the full-scale performance tests shall be performed in accordance with AISI S921.

F. TESTING

F1 General

Tests, when required to determine the strength, flexibility or stiffness of *cold-formed steel structural members* or *connections*, shall be in accordance with *approved* test methods and Section K2.1 of AISI S100 [CSA S136], and shall be conducted under the supervision of a *design professional*.

The following test standards are permitted to be used to determine the strength, flexibility or stiffness of *cold-formed steel structural members* and *connections* via testing:

1. AISI S901, *Test Standard for Determining the Rotational-Lateral Stiffness of Beam-to-Panel Assemblies*
2. AISI S902, *Test Standard for Determining the Effective Area of Cold-Formed Steel Compression Members*
3. AISI S903, *Test Standard for Determining the Uniform and Local Ductility of Carbon and Low-Alloy Steels*
4. AISI S904, *Test Standard for Determining the Tensile and Shear Strengths of Steel Screws*
5. AISI S905, *Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections*
6. AISI S907, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by the Cantilever Test Method*
7. AISI S909, *Test Standard for Determining the Web Crippling Strength of Cold-Formed Steel Flexural Members*
8. AISI S910, *Test Standard for Determining the Distortional Buckling Strength of Cold-Formed Steel Hat-Shaped Compression Members*
9. AISI S911, *Test Standard for Determining the Flexural Strength of Cold-Formed Steel Hat-Shaped Members*
10. AISI S913, *Test Standard for Determining the Strength and Deformation Behavior of Hold-Downs Attached to Cold-Formed Steel Structural Framing*
11. AISI S914, *Test Standard for Determining the Strength and Deformation Behavior of Joist Connectors Attached to Cold-Formed Steel Structural Framing*
12. AISI S915, *Test Standard for Through-The-Web Punchout Cold-Formed Steel Wall Stud Bridging Connectors*
13. AISI S916, *Test Standard for Cold-Formed Steel Framing – Nonstructural Interior Partition Walls With Gypsum Board*
14. AISI S917, *Test Standard for Determining the Fastener-Sheathing Local Translational Stiffness of Sheathed Cold-Formed Steel Assemblies*
15. AISI S918, *Test Standard for Determining the Fastener-Sheathing Rotational Stiffness of Sheathed Cold-Formed Steel Assemblies*
16. AISI S919, *Test Standard for Determining the Flexural Strength and Stiffness of Cold-Formed Steel Nonstructural Members*

13\. AISI S921, *Test Standard for Determining the Strength and Serviceability of Cold-Formed Steel Truss Assemblies and Components*

APPENDIX 1, CONTINUOUSLY BRACED DESIGN FOR DISTORTIONAL BUCKLING RESISTANCE

1.1 Calculation of the *nominal* distortional buckling strength [*resistance*] in flexure in accordance with Section F4 of AISI S100 [CSA S136] is permitted to utilize the restraint provided by *structural sheathing* attached to the compression *flange* of *floor joists* or *ceiling joists* through determination of the rotational stiffness provided to the bending member, k_ϕ of Eq. 1-1. It is also permitted to assume no rotational restraint exists, i.e., $k_\phi = 0$.

1.2 Calculation of the *nominal* distortional buckling strength [*resistance*] in compression in accordance with Section E4 of AISI S100 [CSA S136] is permitted to utilize the restraint provided by *structural sheathing* attached to both *flanges* of *floor joists* or *ceiling joists* through determination of the rotational stiffness provided to the bending member, k_ϕ of Eq. 1-1. It is also permitted to assume no rotational restraint exists, i.e., $k_\phi = 0$.

1.3 The rotational stiffness, k_ϕ, shall be determined in accordance with the following:

$$k_\phi = (1/k_{\phi w} + 1/k_{\phi c})^{-1} \quad (Eq.\ 1\text{-}1)$$

where

$k_{\phi w}$ = *Structural sheathing* rotational restraint

= $EI_w/L_1 + EI_w/L_2$ for interior members (joists or rafters) with *structural sheathing* fastened on both sides (*Eq*. 1-2)

= EI_w/L_1 for exterior members (joists or rafters) with *structural sheathing* fastened on one side (*Eq*. 1-3)

where

EI_w = *Structural sheathing* bending rigidity

= Values as specified in Table 1-1(a) for plywood and OSB

= Values as specified in Table 1-1(b) for gypsum board permitted only for serviceability calculations in accordance with Section L2 of AISI S100 [CSA S136]

L_1, L_2 = One-half joist spacing to the first and second sides respectively, as illustrated in Figure 1-1

$k_{\phi c}$ = *Connection* rotational restraint

= Values as specified in Table 1-2 for fasteners spaced 12 in. o.c. (305 mm) or closer

Table 1-1 (a)[1,2]
Plywood and OSB Sheathing Bending Rigidity, EI_w (lbf-in.^{2}/ft)

Span Rating	Strength Parallel to Strength Axis				Stress Perpendicular to Strength Axis			
	Plywood			OSB	Plywood			OSB
	3-ply	4-ply	5-ply		3-ply	4-ply	5-ply	
24/0	66,000	66,000	66,000	60,000	3,600	7,900	11,000	11,000
24/16	86,000	86,000	86,000	86,000	5,200	11,500	16,000	16,000
32/16	125,000	125,000	125,000	125,000	8,100	18,000	25,000	25,000
40/20	250,000	250,000	250,000	250,000	18,000	39,500	56,000	56,000
48/24	440,000	440,000	440,000	440,000	29,500	65,000	91,500	91,500
16oc	165,000	165,000	165,000	165,000	11,000	24,000	34,000	34,000
20oc	230,000	230,000	230,000	230,000	13,000	28,500	40,500	40,500
24oc	330,000	330,000	330,000	330,000	26,000	57,000	80,500	80,500
32oc	715,000	715,000	715,000	715,000	75,000	615,000	235,000	235,000
48oc	1,265,000	1,265,000	1,265,000	1,265,000	160,000	350,000	495,000	495,000

Note:

1. To convert to lbf-in.^{2}/in., divide table values by 12.
   To convert to N-mm^2/m, multiply the table values by 9.415×10^3.
   To convert to N-mm^2/mm, multiply the table values by 9.415.
2. Plywood and OSB bending rigidity are obtained from APA.

Table 1-1 (b)[1]
Gypsum Board Bending Rigidity
Effective Stiffness (Typical Range), EI_w

Board Thickness (in.) (mm)	EI (lbf-in.^{2}/in.) of width (N-mm^2/mm)
0.5 (12.7)	1500 to 4000 (220,000 to 580,000)
0.625 (15.9)	3000 to 8000 (440,000 to 1,160,000)

Note:
[1] Gypsum board bending rigidity is obtained from the Gypsum Association.

Table 1-2[1]
Connection Rotational Restraint

T (mils)	t (in.)	$k_{\phi c}$ (lbf-in./in./rad)	$k_{\phi c}$ (N-mm/mm/rad)
18	0.018	78	348
27	0.027	83	367
30	0.03	84	375
33	0.033	86	384
43	0.043	94	419
54	0.054	105	468
68	0.068	123	546
97	0.097	172	766

Note:
[1] Fasteners spaced 12 in. (25.4 mm) o.c. or less.

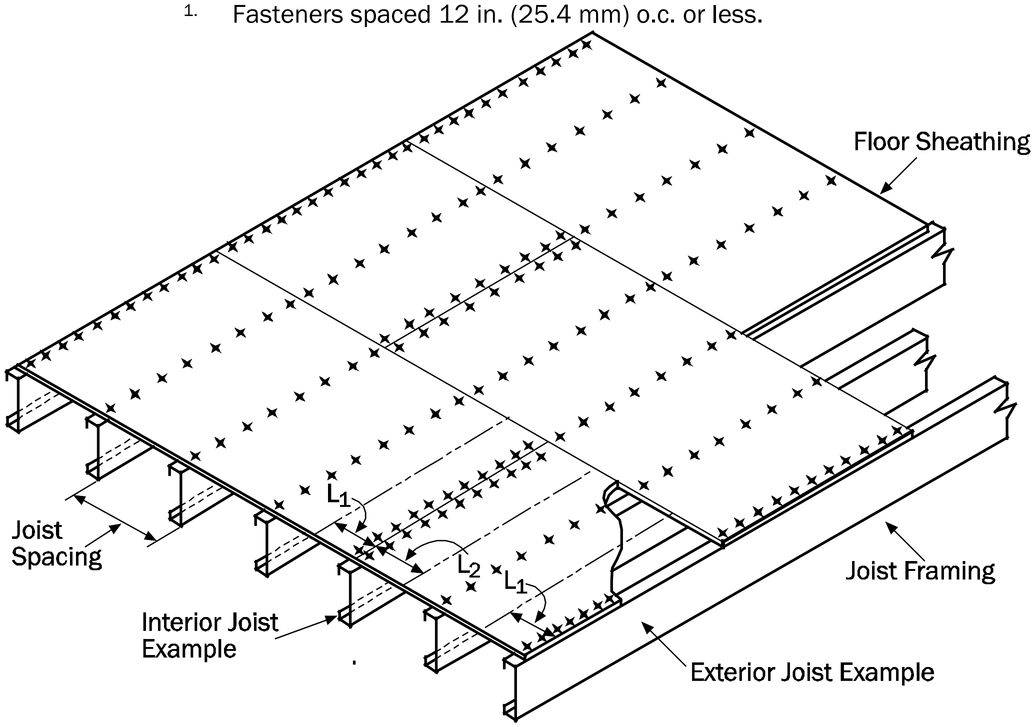

Figure 1-1 Illustration of L_1 and L_2 for Sheathing Rotational Restraint

AISI S240-20-C

AISI STANDARD

Commentary on the

North American Standard for

Cold-Formed Steel

Structural Framing

2020 Edition

DISCLAIMER

The material contained herein has been developed by the American Iron and Steel Institute (AISI) Committee on Framing Standards. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel framing design and installation. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in this *Commentary*.

With anticipated improvements in understanding of the behavior of cold-formed steel framing and the continuing development of new technology, this material will become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a *design professional*. Indeed, in many jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all liability arising therefrom.

1st Printing – December 2020

PREFACE

This *Commentary* is intended to facilitate the use and provide an understanding of the background of AISI S240, *North American Standard for Cold-Formed Steel Structural Framing*. The *Commentary* illustrates the substance and limitations of the various provisions of the Standard.

In the *Commentary*, sections are identified by the same notation as used in the Standard. Words that are italicized are defined in AISI S240. Terms included in square brackets are specific to *Limit States Design* terminology.

American Iron and Steel Institute
November 2020

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COMMENTARY ON THE NORTH AMERICAN STANDARD FOR COLD-FORMED STEEL STRUCTURAL FRAMING

COMMENTARY ON THE NORTH AMERICAN STANDARD FOR COLD-FORMED STEEL STRUCTURAL FRAMING

A. GENERAL

In 2015, AISI S240 (AISI, 2015a) was developed, which included the following previously published standards:

- AISI S200, *North American Standard for Cold-Formed Steel Framing – General Provisions* (AISI, 2012b)
- AISI S210, *North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design* (AISI, 2012d)
- AISI S211, *North American Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI, 2012e)
- AISI S212, *North American Standard for Cold-Formed Steel Framing – Header Design* (AISI, 2012f)
- AISI S213, *North American Standard for Cold-Formed Steel Framing – Lateral Design* (AISI, 2012g)
- AISI S214, *North American Standard for Cold-Formed Steel Framing – Truss Design* (AISI, 2012i)

In 2015, AISI S400, *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems* (AISI, 2015b) was developed. Modifications were made to align the provisions of AISI S240 with AISI S400, as follows:

- The applicability of AISI S240 for seismic design was limited to applications where specific seismic detailing is not required.
- Definitions no longer needed in AISI S240 were removed and remaining definitions were revised, if needed, for consistency with AISI S400.
- Seismic-specific tables for *nominal shear strength* [*resistance*] were deleted.
- Seismic-specific *safety factors* and *resistance factors* were deleted.
- Other seismic-specific requirements were removed, as appropriate, and remaining requirements were generalized for applicability to wind, seismic or other lateral loads.

In 2007, AISI published the first edition of AISI S201 (AISI 2007c), *North American Standard for Cold-Formed Steel Framing – Product Data*, to standardize requirements for cold-formed steel framing products. AISI S201 intended to establish and encourage the production and use of standardized products in the United States, Canada, and Mexico. In 2020, the relevant requirements of AISI S201(AISI, 2017) for *structural members* were incorporated into AISI S240. Within AISI S240, the phrases "standard yield strength", "standard thickness" and "standard shape" are intended to define standard industry practice. In the United States and Mexico, the requirements related to "standard yield strength" and "standard thickness" only become mandatory when required by the construction documents. In Canada, the requirements related to "standard yield strength" and "standard thickness" are mandated by AISI S240. Use of "standard shapes" is not required by AISI S240.

A1 Scope and Applicability

AISI S240 applies to the design and installation of *structural members* utilized in *cold-formed steel light-frame construction* applications and other structures. It applies to floor, wall and roof systems, *lateral force-resisting systems* and *trusses*. However, *cold-formed steel structural members* and *connections* in *seismic force-resisting systems* must be designed in accordance with the additional provisions of AISI S400 where increased seismic performance is required.

The Standard is intended to serve as a supplement to AISI S100 [CSA S136].

Design provisions related to *nonstructural members* can be found in AISI S220, *North American Standard for Cold-Formed Steel Nonstructural Framing* (AISI, 2020b). However, the use of AISI S240 for the design of *nonstructural members* is permitted, since the requirements specified in AISI S240 for *structural members* are equivalent or more stringent than the requirements specified in AISI S220 for *nonstructural members*.

In 2015, the provision that the Standard applies to applications where the specified minimum *base steel thickness* is not greater than 0.1180 inches (2.997 mm) was replaced with the provision that the Standard applies to *light-frame construction* applications, and a definition for the term *light-frame construction* was added to the Standard. *Cold-formed steel structural members* for *light-frame construction* applications are available in a variety of thicknesses. In 2020, standard thicknesses for *cold-formed steel structural members* previously defined in AISI S201 (AISI, 2017) were incorporated in AISI S240.

A2 Definitions

A2.1 Terms

Codes and standards by their nature are technical, and as such specific words and phrases can change the intent of the provisions if not properly defined. As a result, it is necessary to establish a common platform by clearly stating the meaning of specific terms for the purpose of this Standard and other standards that reference it.

In the Standard, *blocking* is defined to transfer shear force or stabilize members. Figures C-A2-1 and C-A2-2 show examples of how *track* or *stud* members are used as *blocking* in various assemblies.

In 2020, the term “repetitive framing” was removed from Section A2.1 because it was not used within the Standard. In addition, having the definition of repetitive framing in the Standard suggested that a 24-inch (610 mm) on center spacing limitation on framing members was applicable to the entire Standard. While *cold-formed steel light-frame construction* can be considered repetitive framing in the sense that common or like members are used at relatively close spacing to provide a framing system, the general scope of the provisions of this Standard are not specific to a maximum member spacing. However, it was determined that the provisions for which the member spacing limitation was applicable should be called out in the specific sections of the Standard.

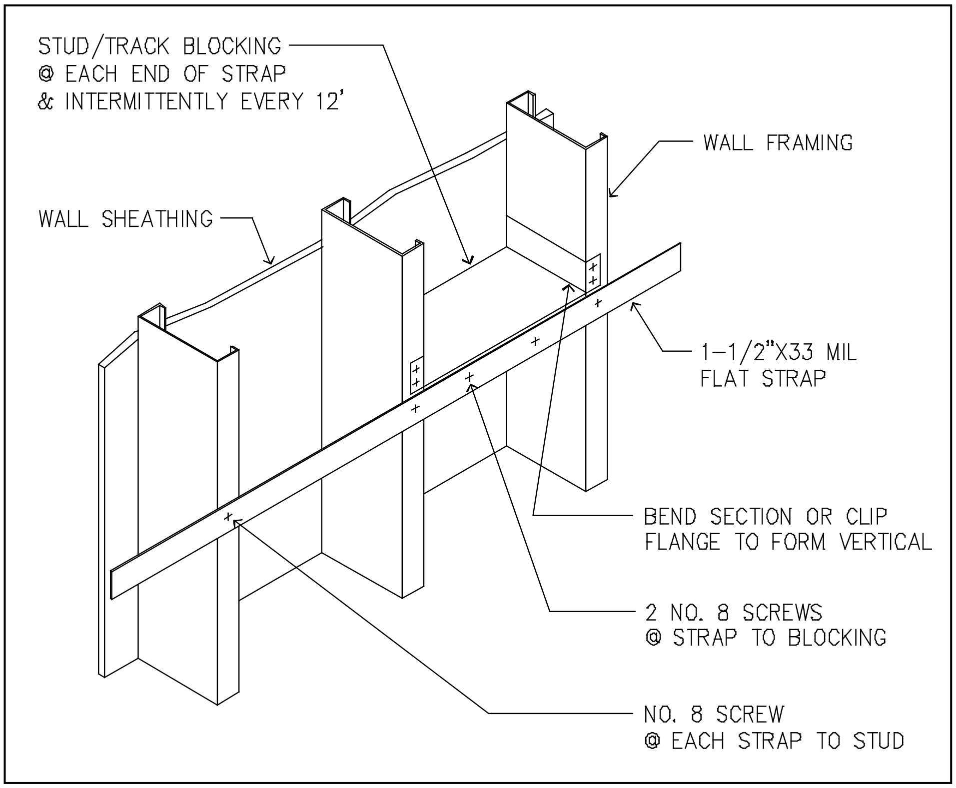


Figure C-A2-1 Application of Stud Track Blocking

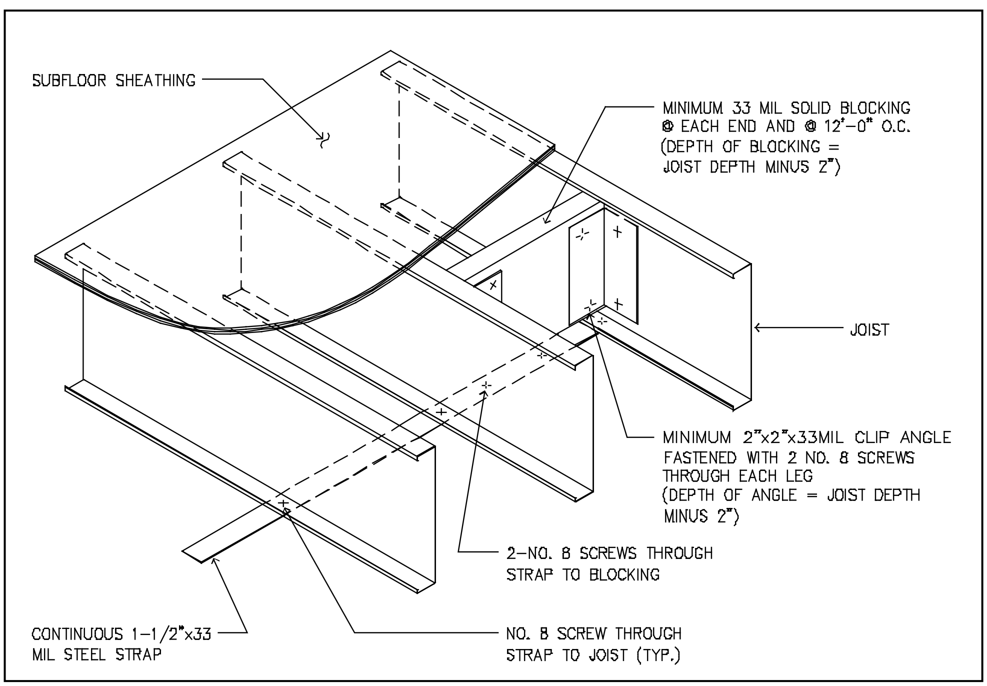


Figure C-A2-2 Application of Joist Blocking

A3 Material

The sheet steel approved for use with this Standard for *structural members* must comply with ASTM A1003/A1003M, except where specifically noted otherwise. ASTM A1003/1003M covers the chemical, mechanical and coating requirements for steel sheet used in the manufacture of *cold-formed steel* framing members such as *studs*, *joists*, and *tracks*.

In 2020, a user note was added to Section A3.1 to highlight that ASTM A1003/1003M is a standard that was developed in order to incorporate requirements for metallic-coated, painted metallic-coated, or painted nonmetallic-coated steel sheet used for *cold-formed steel* framing members into a single standard. Mechanical properties defined in ASTM A1003/A1003M are minimum requirements. For example, the minimum *yield strength* of Type H or L 33 [NS230] steel is 33 ksi [230 MPa]; material with higher *yield strength* is permitted, but 33 ksi [230 MPa] is the design *yield strength*. Additionally, according to the ASTM A1003/1003M standard, Structural Grade Types H and L steel are intended for *structural members* and nonstructural Grade Type NS steel is intended for *nonstructural members*. It is noted that additional country-specific limitations for *curtain wall studs* are provided in AISI S100 [CSA S136], Section A3.2.1.1.

In 2020, Section A3.3 was added to the Standard to clarify that the acceptable steels that may be used for the design of steel *trusses* are defined in Section E4.1.

A4 Corrosion Protection

The minimum coating designations listed in Standard Table A4-1 assume normal exposure and construction practices. Other types of coatings that provide equal or better corrosion protection may also be acceptable. When more severe exposure conditions are probable, consideration should be given to specifying heavier coating weight [mass].

The minimum coating specified by this Standard assumes normal exposure conditions that are defined as having the framing members enclosed within a building envelope or wall assembly within a controlled environment. When more severe exposure conditions are probable, such as industrial atmospheres and marine atmospheres, consideration should be given to specifying a heavier coating. Coating is specified by weight or mass.

This Standard does not require the edges of metallic-coated *cold-formed steel* framing members (shop or field cut in accordance with Standard Section C2.2, punched or drilled) to be touched up with zinc-rich paint, which is able to galvanically protect steel. When base steel is exposed, such as at a cut or scratch, the steel is cathodically protected by the sacrificial corrosion of the zinc coating, because zinc is more electronegative (more reactive) than steel in the galvanic series. A zinc coating will not be undercut because the steel cannot corrode when adjacent to the zinc coating. Therefore, any exposure of the underlying steel at an edge or scratch will not result in corrosion of the steel away from the edge or scratch and thus will not affect the performance of the coating or the steel structure (CFSEI, 2007a).

It is noted that ASTM A1004/A1004M (ASTM, 2018) covers procedures for establishing the acceptability of steel sheet for use as *cold-formed steel* framing members. This practice is to be used to assess the corrosion resistance of different coatings on steel sheet in a laboratory test. It is not intended to be used as an application performance standard for the *cold-formed steel* framing, but is to be used to evaluate coatings under consideration for addition to ASTM A1003/A1003M.

Direct contact with dissimilar metals (e.g., copper, brass, etc.) should be avoided in order to prevent unwanted galvanic action from occurring. Methods for preventing the contact from occurring may be through the use of nonconductive and noncorrosive grommets at *web* penetrations or through the use of non-metallic brackets (a.k.a. isolators) fastened to hold the dissimilar metal building products (e.g., piping) away from the *cold-formed steel* framing. In 2006, a change was made allowing the use of dissimilar metals in contact with *cold-formed steel* framing, provided the specific application is approved. It was recognized that dissimilar metals in contact with *cold-formed steel* framing might not always be a problem. For example, there are no galvanic concerns where there is no moisture. A special case of dissimilar metals occurs in Canada where, for certain climatic conditions and building heights, the use of stainless steel brick ties is required. When these ties are connected to steel *stud* backup, contact between dissimilar metals can occur. For guidance on this issue of dissimilar metals, refer to the Canadian standard CAN/CSA-A370-14, *Connectors for Masonry* (CSA, 2014).

When there is direct contact of *cold-formed steel* framing with pressure-treated wood, the treated wood, *cold-formed steel* framing, connector and/or fastener manufacturers should be contacted for recommendations. Methods that should be considered may include specifying a less corrosive pressure treatment (sodium borate, organic preservative systems, etc.), isolating the *cold-formed steel* and wood components, or changing details to avoid use of pressure-treated wood altogether.

Design professionals should take into account both the initial contact with wet or damp building materials, as well as the potential for those materials to absorb water during the building's life, as both circumstances may accelerate corrosion.

In 2007, the Cold-Formed Steel Engineers Institute updated the 2004 AISI document, entitled *Durability of Cold-Formed Steel Framing Members* (CFSEI, 2007a), to give engineers, architects, builders and homeowners a better understanding of how galvanizing (zinc and zinc alloy coatings) provides long-term corrosion protection to *cold-formed steel* framing members. Additional information can be obtained from the American Galvanizers Association publication entitled *Hot Dip Galvanizing For Corrosion Protection—A Specifier's Guide* (AGA, 2012) and the Cold-Formed Steel Engineers Institute's publication entitled *Corrosion Protection for Cold-Formed Steel Framing in Coastal Areas* (CFSEI, 2007b).

A5 Products

AISI S100 [CSA S136] permits the minimum delivered *base steel thickness* (exclusive of any coatings) of a *cold-formed steel* member to be 95% of the *design thickness*. This Standard, therefore, specifies the minimum *base steel thickness* that complies with AISI S100 [CSA S136]. The thickness designations are consistent with standard industry practice, as published previously in AISI S201 (AISI, 2017) and as incorporated in AISI S240 in 2020. It is recommended that thickness measurements be taken in the middle of the flat of the *flange* or *web* of the cross-section.

Section A5.3 has adopted a standard designator system for identifying *cold-formed steel* framing members. The intent for using a standard designator system was to overcome the varied designators that were produced by each individual manufacturer. In addition, the designator is used to identify not only a specific *cold-formed steel* framing member, but also to identify the section properties of that same member through the use of the manufacturer's product technical information documents.

The following presents an example of the standard designator for a *cold-formed steel stud*:

350S162-33 represents a member with the following:

350S162-33

33 for 33 *mil* (0.0329 inch) (0.836mm) *designation thickness*

162 for 1.625 inch (41.3 mm) *flange* width

"S" for *C-shape*

350 for 3.50 inch (89.9 mm) *web* depth

In 2011, as part of an exercise to synchronize all relevant codes and specifications, the minimum tolerances for the manufacture of *cold-formed steel* framing members were included in the AISI framing standards. The minimum tolerances for the manufacture of *structural members* can be found in Section A5.4. In 2014, manufacturing tolerances for stiffening lip length and *flange* width were also added. The revisions are consistent with ASTM C955 (ASTM, 2011c). The minimum tolerances for the manufacture of *nonstructural members* can be found in AISI S220(AISI, 2020b). The manufacturing tolerances for length, *web* width, camber, bow, twist, etc. of framing members are consistent with ASTM C955, ASTM C645 (ASTM, 2011b), and manufacturers' certification programs.

To aid in shop and field verification, all framing members are to carry a product identification to indicate conformance with the minimum *base steel thickness*, coating designation, minimum *yield strength*, and manufacturer's name.

In 2011, color coding of individual framing members or groups of like members were removed from the AISI framing standards with consideration that the color coding approach could cause confusion in differentiating between *structural* and *nonstructural members* of the same thickness. Further, color coding is optional and the criterion, if needed, exists in a non-mandatory Appendix of ASTM C645.

In 2020, the product standards in AISI S201 were incorporated into this Standard. In addition, the second part of the four-part product designator was changed to clarify that the various member types can be used in variety of applications.

A5.2 Minimum Flange Width

In 2012, as part of an exercise to synchronize all relevant codes and specifications, the provisions for minimum *flange* width were added to the AISI framing standards, and were consistent with ASTM C645 (ASTM, 2011b) and ASTM C955 (ASTM, 2011c). The minimum *flange* width for *C-shape* members was included in the Standard to accommodate a butt joint of sheathing. The minimum *flange* width for *track* members was included in the Standard to accommodate an edge joint of sheathing.

B DESIGN

B1 General

B1.1 Loads and Load Combinations

Currently, ASCE 7 has no geographical-based information on Mexico. Therefore, users with projects in Mexico should work with the appropriate *authority having jurisdiction* to determine appropriate loads and load combinations that are consistent with the assumptions and rationale used by ASCE 7.

In 2009, the words "with no axial loads" were deleted to clarify that the intent is to evaluate deflections for bending of the wall *stud* alone when subjected to the Components and Cladding (C&C) wind loads.

B1.1.1 Live Load Reduction on Wall Studs

Since some building codes allow designers to reduce floor live load as a function of area of floor supported by gravity load-bearing members, a requirement was added in 2009 to clarify appropriate application of area live load reduction for the design of individual *structural members* in load-bearing wall framing, including vertical members such as *studs* or columns and horizontal members such as *headers* or beams within a wall assembly. The floor area (from one or more floors) contributing load to a wall framing member should be determined in a manner consistent with engineering mechanics.

B1.1.2 Wind Loading Considerations in the United States and Mexico

Because a wall *stud* subject to combined bending and axial load resists wind loads imposed on two surfaces, the member can be analyzed based on Main Wind Force Resisting System (MWFRS) wind loads. For bending alone, the wall *stud* experiences wind from only one surface and therefore must be analyzed for Components and Cladding (C&C) wind loads.

Section 1609.6.2.3 of the *International Building Code* (ICC, 2003) states that:

"*Members that act as both part of the main force resisting system and as components and cladding shall be designed for each separate load case.*"

Discussion in the *Southern Standard Building Code* Commentary (SBCCI, 1999) sheds light on a reasonable approach to the design of wall *studs* for wind resistance, stating that:

"*Some elements of a building will function as part of the main wind force resisting system and components and cladding also. Such members include but not limited to roof panels, rafters, and wall studs. These elements are required to be designed using the loads that would occur by considering the element as part main wind force resisting system, and also separately checked or designed for loads that would occur by considering the element as component and cladding. The use of this section can be demonstrated by considering, for example, the design of a wall stud. When designing the stud for main wind force resisting system loads, all loads such as bending from the lateral force with the wind on the wall in addition to any uplift in combinations with the dead load of the roof or a story above induced by the simultaneous action of roof forces should be considered together. When designing the stud for component and cladding loads, only the bending resulting from the wind force normal to the stud and the dead load associated with that member should be considered. The member should be sized according to the more critical loading condition.*"

The wood industry has also investigated this condition and has adopted a similar policy as shown in the *Wood Frame Construction Manual* (AFPA, 1995), where Section 2.4 states that:

"*Studs tables are based upon bending stresses induces by C&C Loads. The bending stresses are computed independent of axial stresses. In addition, the case in which bending stresses from MWFRS loads act in combination with axial stresses from wind and gravity loads have been analyzed. For buildings limited to the conditions in the WFCM-SBC, the C&C loads control stud design.*"

The Commentary to Appendix C of ASCE 7 (ASCE, 2006) provides some guidance on the selection of loads for checking the serviceability limit state of buildings and their components, where Section B1.2 states in part:

"*Use of factored wind load in checking serviceability is excessively conservative. The load combination with an annual probability of 0.05 of being exceeded, which can be used in checking short-term effects, is D + 0.5L + 0.7W.*"

Thus, using 70% of the wind load from Components and Cladding for checking deflections should conservatively satisfy the above. In 2012, IBC Table 1604.3, Footnote f recommended that 42% of the wind load be used for checking deflections. The 2012 IBC is based on ASCE 7-10 and the ASD wind load factor of 0.6W is used to arrive at a service load pressure; thus, $0.7 \times 0.6 = 0.42$.

AISC Design Guide No. 3 (Fisher and West, 1990) also recommends reduced wind loads when checking the serviceability of cladding based upon a 10-year return period or 75 percent of the 50-year wind pressure.

B1.2 Design Basis

The strength determinations required by this Standard are to be in accordance with AISI S100 [CSA S136]. For design guidance on the application of AISI S100 [CSA S136] to typical *cold-formed steel* construction, refer to *Design Guide for Cold-Formed Steel Framing* (AISI, 2016) and *Cold-Formed Steel Design* (Yu, LaBoube and Chen, 2020).

B1.2.1 Floor Joists, Ceiling Joists and Roof Rafters

The Standard permits the design of *floor joists*, *ceiling joists* and *roof rafters* to be based on either a discretely braced design in which discrete braces are provided along the member's length, or based on a continuously braced design in which attached sheathing or deck are attached in accordance with the Standard.

The continuously braced design provisions of the Standard are limited to *floor joists*, *ceiling joists* and *roof rafters* with dimensions and properties that are within the range of standard products, as defined in this Standard. This limitation was deemed appropriate due to the availability of research and field experience with such members.

B1.2.2 Wall Studs

The Standard permits the design of wall *studs* to be based on either an all steel design in which discrete braces are provided along the member length, or based on a sheathing braced design. It is permitted by this Standard to use sheathing attached to both *flanges,* or to use a combination of sheathing attached to one *flange* and discrete bracing stabilizing the other *flange*. Because load-bearing wall *studs* used in multi-story construction may go unsheathed for an extended period of time during construction, it is uncommon to use a sheathing braced design approach for these applications.

The 2007 edition of AISI S100 (AISI, 2007a) added new design provisions for considering distortional buckling of *cold-formed steel* members in bending and compression. In 2009, distortional buckling was introduced in this Standard, and separate provisions provided for discretely braced design and continuously braced design. Detailed technical information on distortional buckling can be found in the *Commentary* on AISI S100 (AISI, 2020a).

a) Flexural Strength

When the *curtain wall stud* has sheathing only on one *flange* and discrete bracing on the other *flange*, the *available flexural strength* [*factored resistance*] is the minimum strength considering lateral-torsional buckling, local buckling and distortional buckling determined in accordance with the provisions of Chapter F of AISI S100 [CSA S136]. When sheathing is attached to the compression *flange*, the lateral-torsional buckling does not need to be considered, i.e., the *available flexural strength* [*factored resistance*] is determined by considering local buckling assuming $F_n = F_y$ or $M_{ne} = M_y$ in Section F3 of AISI S100 [CSA S136]. Distortional buckling should be considered whether or not the sheathing is attached to the compression *flange*. Discrete braces for restraining distortional buckling must restrict rotation at the *web*/*flange* juncture. This may be accomplished with sufficiently stiff *blocking* to restrict rotations in the *web*, with *strap* across a series of members lacing from compression *flange* of one member to the tension *flange* of the next member and continuing, or with other systems – engineering judgment is required. Provisions for continuously braced design for distortional buckling are provided in Section B2.6.

In 2015, the maximum discrete bracing spacing of 8 ft (2.44 m) on center is required based on design experience and common practice for flexural members such as *curtain wall studs*.

The Standard stipulates that when sheathing braced design is used, the wall *stud* shall be evaluated without the sheathing *bracing* for the dead loads and loads that may occur during construction, or in the event that the sheathing has been removed or has accidentally become ineffective. In 2014, the LRFD load combination for the United States and Mexico was taken from ASCE 7 (ASCE, 2006) for special event loading conditions.

b) Compression Strength

Although the design approach for sheathing braced design is based upon engineering principles, the Standard limits the sheathing braced design to wall *stud* assemblies, assuming that identical sheathing is attached to both sides of the wall *stud*. This limit recognizes that identical sheathing will aid in minimizing the twisting of the section. If only single-sided sheathing is used, additional twisting of the section will occur, thus placing a greater demand on the sheathing; therefore, the *stud* must be designed and braced as an all steel assembly.

The provision that wall *studs* with sheathing attached to both sides that is not identical is permitted to be designed based on the assumption that the weaker of the two sheathings is attached to both sides is based on engineering judgment. Determination of which of the two sheathings is weaker should consider the sheathing strength, sheathing stiffness and sheathing-to-wall *stud connection* capacity, as applicable.

B1.2.3 In-Line Framing

In-line framing is the preferred and most commonly used framing method. The advantage of in-line framing is that it provides a direct load path for transfer of forces from *joists* to *studs*. The Standard stipulates maximum framing alignment to minimize secondary moments on the framing members. Weak axis bending strength of *track* is minimal, and therefore the *track* cannot function as a load transfer member. In the absence of in-line framing, a load distribution member, such as a structural *track*, may be required for this force transfer.

Industry practice has accepted in-line framing to mean that the *joist*, *rafter*, *truss* and structural wall *stud* framing would be aligned so that the center line (mid-width) is within ¾ inch (19 mm) of the center line (mid-width) of the load-bearing members beneath. However, the ¾ inch allowable offset creates the possibility for a misalignment in the load path from an upper story load bearing *stud* wall, through a *joist* with a *bearing stiffener*, and onto a load-bearing *stud* or foundation wall below. In 2003, a total of 110 end- and interior-two-*flange* loading tests of various floor *joist* assemblies were carried out at the University of Waterloo (Fox, 2003) to determine the effect that an offset loading has on the strength of typical floors. It was concluded that an additional limit should be placed on the *bearing stiffener* offset to the load-bearing members above or beneath for cases where the *bearing stiffener* is attached to the back of the *joist* as depicted in Figure B1.2.3-1.

As an alternative to in-line framing, the Standard permits the use of a structural load distribution member that is specified in accordance with an *approved* design or *approved* design standard. As an aid to designers, strength and stiffness have been determined experimentally for various load-bearing top *track* assemblies (NAHB-RC, 2003; Dawe, 2005), including standard steel *track*, deep-leg steel *track*, and steel *track* with a 2x wood top plate. Design guidance for some of the typical top *track* load distribution members is available from the Cold-Formed Steel Engineers Institute (CFSEI, 2010a).

B1.2.6 Principles of Mechanics

The Standard does not aim to limit *cold-formed steel* light-framed *shear walls*, *strap braced walls* and *diaphragms* to the configurations included in the Standard. As such, the development of design values for other systems or configurations is permitted in accordance with rational engineering procedures and principles of mechanics. Design values based on calculations must, however, recognize the fundamental differences between the expected performance of structures under wind and seismic *loads*, and the performance of an individual lateral element. It must also be recognized that the tabulated design values in the Standard are based on test data for individual lateral elements. Recognition of these differences requires, where appropriate, that calculated values be scaled per existing design data. For wind design, there is no modification in design loads per the lateral resisting system used.

B1.3 Built-Up Sections

Section I1.2(b) of AISI S100 [CSA S136] provides a prescriptive requirement for built-up compression members to preclude any shear slip in a built-up cold-formed steel section.

In order to determine the applicability of this prescriptive end *connection* requirement for double back-to-back *cold-formed steel studs*, a series of tests were conducted at the University of Missouri-Rolla (UMR). This research by Stone and LaBoube (2005) demonstrated that when a built-up *stud* is seated in a *track* section and bears on a firm surface, end slip is precluded; and thus the need for the additional fasteners is not required. The UMR research consisted of 32 tests of two C-shaped sections with edge stiffened *flanges*. The built-up sections tested were 82.6 inches (2.1 m) long, between 3-5/8 inches (92 mm) and 6 inches (152 mm) in depth, and between 33 mils (0.84 mm) and 54 mils (1.37 mm) thick. The *studs* were attached back-to-back with two rows of screws through the *web* starting 2 inches (50 mm) from each end and spaced at 12 inches (305 mm), 24 inches (610 mm) or 36 inches (914 mm) on-center. Each end of the built-up *stud* section was screwed to a standard *track* section 12 inches long which in turn was bearing on a 6″× 8″× ½″ (152 mmm × 203 mm × 12.7 mm) steel bearing plate. In order to simulate a pin *connection* representing an effective length factor of unity, the steel bearing plate at each end was supported on a round bar.

In order to justify not specifying the end *connection* requirement of Section I1.2(2) of AISI S100 [CSA S136], the designer must meet the bearing condition of support on steel or concrete to ensure relative slip of the two sections is prevented. This requires that the *track* must be supported over the entire area of the built-up member by steel or concrete components and that bearing stresses on the steel or concrete are within allowable limits.

B1.5 Connection Design

Self-drilling screws are the primary fastener type used in cold-formed steel construction, although the Standard does not preclude the use of other fastener types.

B1.5.1 Screw Connections

B1.5.1.1 Steel-to-Steel Screws

ASTM C1513 covers screws for the *connection* of *cold-formed steel* members manufactured in accordance with ASTM Specifications C645 and C955. This specification also covers test methods for determining performance requirements and physical properties. However, the tensile or shear strength must be determined by test in accordance with AISI S904. General guidance on the selection of screws is given by the Cold-Formed Steel Engineers Institute document: *Screw Fastener Selection for Cold-Formed Steel Frame Construction* (CFSEI, 2011).

Proper selection and installation of screws are necessary to ensure the design performance. Screws are specified using a nominal size designator, not by diameter. Table C-B1.5.1.1-1 defines suggested nominal screw diameters. The installation requirements stated in this Standard are based on industry practice. Selection of a minimum screw size is based on the total sheet thickness of the *connection*. Where recommendations are not available, Table C-B1.5.1.1-2 provides suggested screw size for steel-to-steel *connections* as a function of point style, per ASTM C1513, and total combined thickness of all connected steel members.

Table C-B1.5.1.1-1
Suggested Screw Body Diameter

Screw Nominal Size	Nominal Screw Diameter, d	
	(inches)	(mm)
No. 6	0.138	3.51
No. 8	0.164	4.17
No. 10	0.190	4.83
No. 12	0.216	5.49
1/4”	0.250	6.35

Table C-B1.5.1.1-2
Suggested Screw Sizes for Steel-to-Steel Connections

Screw Size	Point Style	Total Thickness of Steel [1]	
		(inches)	(mm)
¼”	1	0.024 – 0.095	0.61 – 2.41
No. 6	2	0.036 – 0.100	0.91 – 2.54
No. 8	2	0.036 – 0.100	0.91 – 2.54
No. 10	2	0.036 – 0.110	0.91 – 2.79
No. 12	2	0.050 – 0.140	1.27 – 3.56
No. 14	2	0.060 – 0.120	1.52 – 3.05
No. 18	2	0.060 – 0.120	1.52 – 3.05
No. 8	3	0.100 – 0.140	2.54 – 3.56
No. 10	3	0.110 – 0.175	2.79 – 4.45
No. 12	3	0.090 – 0.210	2.29 – 5.33
No. 14	3	0.110 – 0.250	2.79 – 6.35
No. 12	4	0.175 – 0.250	4.45 – 6.35
¼”	4	0.175 – 0.250	4.45 – 6.35
No. 12	4.5	0.145 – 0.312	3.68 – 7.92
No. 12	5	0.250 – 0.500	6.35 – 12.7
¼”	5	0.250 – 0.500	6.35 – 12.7

[1] Combined thickness of all connected steel members

B1.5.1.3 Edge Distance, End Distance and Spacing

To be consistent with AISI S100 [CSA S136], the minimum edge distance requirement was revised to 1.5 times the screw nominal diameter in 2020.

AISI S100 [CSA S136] stipulates that the center-to-center spacing of screws be at least 3 times the screw diameter. During installation, if this spacing is only 2 times the diameter, research at the University of Missouri-Rolla (Sokol et al., 1999) has shown that the structural performance of the *connection* is reduced. Guidelines for center-to-center spacing of less than 2 times the diameter are not stipulated because the screw head diameter precludes a smaller spacing. The University of Missouri-Rolla research serves as the basis for the requirements in the Standard.

B1.5.1.4 Gypsum Board

The Standard employs the use of the *applicable building code* as the guide for provisions that cover the installation and attachment of gypsum panels to *cold-formed steel* framing. The model building codes in the United States reference ASTM C1007 (ASTM, 2020) as the appropriate standard for the gypsum board attachment to *cold-formed steel structural members*.

The Standard requires that screw fasteners for gypsum board to steel *connections* be in compliance with ASTM C954, ASTM C1002 or ASTM C1513, as applicable, with a bugle head style. ASTM C954 is for fastening to steel having a thickness from 0.033 inches (0.84 mm) to 0.112 inches (2.84 mm). ASTM C1002 is for fastening to steel with a thickness less than 0.033 inches (0.84 mm). ASTM C1513 is for fastening to steel with a thickness not greater than 0.118 inches (2.997 mm).

B1.5.2 Welded Connections

Additional guidance on welding of *cold-formed steel* members is provided in the Cold-Formed Steel Engineers Institute document *Welding Cold-Formed Steel* (CFSEI, 2010b).

B1.5.3 Bolts

The Standard permits the use of bolts. Bolted *connections* can be designed by AISI S100 [CSA S136] equations. See the *Commentary* on AISI S100 (AISI, 2020a) for background information.

B1.5.4 Power-Actuated Fasteners

The Standard permits the use of power-actuated fasteners. *Connections* using power-actuated fasteners can be designed by AISI S100 [CSA S136] equations. See the *Commentary* on AISI S100 (AISI, 2020a) for background information.

B1.5.5 Other Connectors

The Standard permits the use of proprietary fasteners such as pneumatically driven pins, rivets, adhesives, and clinches. Proprietary fasteners must be designed and installed in accordance with the manufacturers' requirements. The *safety and resistance factors* to be used in design is to be determined by Section K2.1 of AISI S100 [CSA S136]. The Cold-Formed Steel Engineers Institute has published technical notes pertaining to power-actuated fasteners and pneumatically driven pins (CFSEI, 2012; CFSEI, 2009).

B1.6 Web Crippling

The *web* crippling strength [*available resistance*] can be determined using the design provisions provided in AISI S100. *Commentary* to AISI S100 (AISI, 2020b) provides background information for those design provisions.

An exception was added in 2020 for C-Section with *web*-to-thickness ratio (h/t) greater than 200 but less than or equal to 260. These provisions for *web* crippling strength were developed based on test data that included h/t ratios as large as 259 (Beshara and Schuster, 2000a). Although the test data (Hetrakul and Yu, 1980) was for both end-one-*flange* loading and interior-one-*flange* loading with the *flange* unfastened to the support, the 5% reduction required of Section B1.6 is to address potential unconservatism in the design equation for the higher h/t ratios.

The limit of $e_h = 3.9\ d_h$ was derived by setting AISI S100 Equation G6-1 equal to 1.0 and solving for x.

B2 Floor and Ceiling Framing

B2.2 Floor Joist Design

B2.2.1 Bending

Floors composed of repetitively framed *floor joists* with nominally identical members, multiple attachment points to a *structural sheathing* or *steel deck*, and the ability to redistribute under load have been shown to have system reliability which greatly exceeds the component reliability of the individual *floor joist* (Smith et al. 2016, 2018). This Standard allows the designer to account for some of the benefit of this increased reliability by increasing the *available strength* [*factored resistance*] of the *floor joist* by 15% when specific conditions are met. The design of the *floor joists* may follow discretely braced design, or continuously braced design, but even for discretely braced design the conditions of this section must be met for the 15% system reliability benefit to be utilized.

Related to condition (i) in Standard Section B2.2.1, it has been shown that when nominally identical *floor joists* are used in a floor, for example for construction efficiency, the demand/capacity ratio varies greatly across the floor and the resulting floor system has higher reliability than its weakest member (Smith et al. 2016). The greater the floor width and *floor joist span* the more pronounced this benefit becomes; the 12-ft (3.7-m) width is based on the building studied in Smith et al. (2016) and practical considerations for panelized floors.

Related to condition (ii) in Standard Section B2.2.1, the *structural sheathing* or *steel deck* must be capable of stabilizing the compression *flange* of the *floor joist* and re-distributing load amongst the *floor joists* for the system reliability benefits from redistribution detailed in Smith et al. (2018) to be realized. The sheathing requirements for continuously braced design were selected to provide this beneficial redistribution, but *structural sheathing* or *steel deck* solutions with greater stiffness and strength are allowed. It has been shown that when *structural sheathing* has many attachment points to a *structural member* the in-plane shear resistance of the system has increased reliability (Bian et al., 2017; Chatterjee et al., 2018). Redistribution of load amongst *floor joists* relies on this mechanism, thus condition (b) also requires a minimum fastener spacing. The minimum fastener size is based on the continuously braced design requirements. The Standard recognizes that other fasteners may be utilized, but requires a minimum spacing and stiffness and strength that at least exceeds that of a No. 8 screw. In the studies of Smith et al. (2018) the *structural sheathing* was 8 ft (2.4 m) long and placed transverse to the *floor joists* and thus connected to 5 *floor joists*, spaced 2 ft (0.6 m) o.c. Recognizing that all floors cannot be broken into 8-ft (2.4-m) spans, and that the most important redistribution is to the nearest neighbor *floor joist,* the Standard requires that the *structural sheathing* or *steel deck* span at least 3 *floor joists*.

Related to condition (iii) in Standard Section B2.2.1, if the tension *flanges* of the *floor* *joists* are not restrained, then unfavorable section twist and distortion about the top *flange* will occur and the *floor joists* will not be able to properly redistribute as necessary for improved system reliability. The Standard allows this torsion restraint to be designed, but also specifies a minimum acceptable solution consisting of *straps* and *blocking*. The criteria for the *straps* and *blocking* are parallel to those found in AISI S230 (AISI, 2019) but here only define minimal acceptable details and defer to the strength requirements of Standard Section B2.6(2) or those of AISI S100 Section C2.2, which must still be satisfied.

Related to condition (iv), the Standard requires solid blocking consistent with Figure C-A2-2. The floors studied in Smith et al. (2016, 2018) and Chatterjee et al. (2018) all had solid *blocking*. These additional minimal requirements ensure that individual *floor joists* are stabilized, may redistribute load, and act together as part of a larger floor assembly thus insuring improved system reliability.

During development of this Standard criteria for the cross-section slenderness of the *floor joists* was considered. Ayan and Schafer (2017) quantified the manner in which cross-section slenderness may be used to determine moment-rotation behavior of *floor joists* and Smith et al. (2018) shows that the potential system benefit is maximized when moment-rotation response of the *floor joists* is elastic-plastic. The Standard chooses to only utilize a small portion of the system reliability benefit identified in Smith et al. (2018) and to primarily focus on the benefits realized from repetitive members and torsionally stable assemblies with multiple attachment points between *floor joists* and *structural sheathing* or *steel deck*. For *floor joists* with cross-section slenderness that allows for development of inelastic reserve, the potential system reliability benefits are even greater than utilized by this Standard.

B2.5 Bearing Stiffeners

The Standard provides provisions for clip angle bearing stiffeners, based on research at the University of Waterloo (Fox, 2006).

B2.6 Bracing Design

The continuous bracing and *flange* bracing provisions of the Standard were deemed appropriate due to the availability of research and field experience with such assemblies. The requirements in Section B2.6 were adapted from AISI S100 [CSA S136] requirements for members where neither *flange* is attached to sheathing.

B3 Wall Framing

B3.2 Wall Stud Design

B3.2.1 Compression

Prior to 2004, the *North American Specification for the Design of Cold-Formed Steel Structural Members* contained requirements for sheathing braced design in its Section D4(b). In 2004, these provisions were removed. AISI S100 [CSA S136] stipulates that *structural members* utilized in *cold-formed steel* repetitive framing applications be designed in accordance with AISI S240.

Sheathing braced design in the Standard is based on rational analysis assuming that the sheathing braces the *stud* at the location of each sheathing-to-*stud* fastener location. Axial load in the *stud* is limited, therefore, by the capacity of the sheathing or sheathing-to-wall *stud connection*. Using the bracing principles as defined by Winter (1960) and summarized by Salmon and Johnson (1996), the equilibrium of an imperfect column braced at mid-height, as illustrated in Figure C-B3.2.1-1, is expressed as:

$$P(\Delta+\Delta_0) = k\frac{\Delta}{2}\frac{L}{2} \qquad \text{(C-B3.2.1-1)}$$

where

Δ = Sidesway deflection at brace location beyond initial imperfection, Δ_0

Δ_0 = Initial imperfection

L = Total *stud* height

k = Bracing stiffness

Following Eq. C-B3.2.1-1 to solve for deflection, Δ:

$$\Delta = \frac{P\Delta_0}{\frac{kL}{4} - P}$$

When P approaches kL/4, the lateral deflection approaches infinity. Therefore, P_{cr}=kL/4. The bracing stiffness corresponding to P_{cr} is called k_{ideal}, and $k_{ideal} = 4P_{cr}/L$. To provide sufficient bracing to a column subjected to load P, k needs to be greater than k_{ideal}. It is common to choose $k = 2k_{ideal} = 2(4P/L)$, and the corresponding deflection is:

$$\Delta = \frac{P\Delta_0}{\frac{kL}{4} - P} = \frac{P\Delta_0}{2\left(\frac{4P}{L}\right)\frac{L}{4} - P} = \Delta_0$$

For $\Delta_0 = L/960$, the bracing force is

$$F_{br} = k\Delta = 2\frac{4P}{L}\Delta_0 = 0.8\%P$$

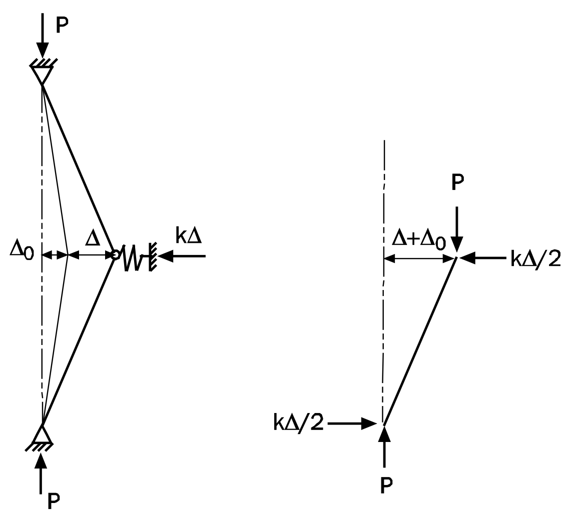


Figure C-B.2.1-1 Column Lateral Bracing

This result is consistent with the requirement of 1%P from AISI S100 [CSA S136] Equation C2.3-1. In this Standard, a more conservative approach has been taken by considering a safety factor of 2, which results in F_{br} = 2%P. Since the tests indicated a failure of the sheathing, not the screw-to-*stud* attachment, the Standard does not directly stipulate a design requirement to check the screw-to-*stud* capacity or the screw capacity in shear.

The strength of gypsum sheathing attached with No. 8 and No. 6 screws is based on tests by Miller (1989) and Lee (1995), respectively. Based on engineering judgment, a factor of safety of 2.0 was applied to the ultimate load per screw (Table C-B3.2-1) when determining the brace strength per screw for the gypsum wallboard. The ultimate loads are based on the averaging of test data provided in Miller (1989) and Lee (1995).

Table C-B3.2-1
Brace Strength [Resistance] Per Screw
Limited by Gypsum Sheathing-to-Wall Stud Connection Capacity

Sheathing	Screw Size	Ultimate Load (per screw)	Brace Strength [Specified](per screw)
1/2 inch (12.7 mm)	No. 6	0.117 kips (0.516 kN)	0.058 kips (0.258 kN)
1/2 inch (12.7 mm	No. 8	0.134 kips (0.596 kN)	0.067 kips (0.298 kN)
5/8 inch (15.9 mm)	No. 6	0.136 kips (0.605 kN)	0.068 kips (0.302 kN)
5/8 inch (15.9 mm)	No. 8	0.156 kips (0.694 kN)	0.078 kips (0.347 kN)

The maximum axial nominal load provided in Table B3.2-1 of the Standard was determined by first multiplying the brace strength [*resistance*] per screw given in *Commentary* Table C-B3.2-1 by 2 to recognize that sheathing must be attached to both *stud flanges* and then dividing by 0.02 (i.e., 2%P), thus solving for the nominal load P that is given in Table B3.2-1 of the Standard.

The unbraced length with respect to the minor axis and the unbraced length for torsion are taken as twice the distance between the sheathing connectors in the event that an occasional attachment is defective to a degree that it is completely inoperative.

If plywood sheathing is attached to both *flanges* of the wall *stud* by screws spaced no greater than 12 inches (305 mm) on center, both the plywood and the *stud* must be checked. The following outlines a possible design solution for plywood attached to a wall *stud*:

Evaluation of the Plywood:

Using NDS (AFPA, 1997) Section 11.3,

Nominal Design Value (Brace Strength), $Z = D\,\ell_m\,F_{em}/R_d$

D = 0.164″ (4.17 mm) (No. 8 Screw)

ℓ_m = Sheathing thickness = 0.5″ (12.7 mm)

R_d = 2.2 for $D \le 0.17''$ (4.32 mm)

F_{em} = 1900 psi (13,100 kPa) (lowest bearing strength value – the values are based on the specific gravity of the wood)

Z = 0.164 × 0.5 × 1900 / 2.2 = 70.82 lbs. (315 N)

Brace Force, $F_{br} = 0.02\,P$, where P is the axial load in the *stud*.

P = 70.82/0.02 = 3,540 lbs (15,700 N) = 3.5 kips (15.6 kN) per screw x 2 screws = 7.0 kips (31.1 kN) per *stud*

Evaluation of the Steel Wall *Stud*:

The screw capacity in the *stud* can be evaluated using Section J4.3 of AISI S100 [CSA S136],

where

$P_{ns} = 4.2\,(t^3d)^{0.5}\,F_u \le 2.7\,tdF_u$

Ω = 3.0

If $P_{ns}/\Omega < Z$, the brace force analysis to determine P should be based on the lower value. The capacity per screw bracing is computed by the following equation:

$P = (P_{ns}/\Omega < Z)/0.02$

Because the Standard requires that sheathing must be attached to both *flanges* of the wall *stud*, the *nominal* axial load in the wall *stud* is twice the value of P.

B3.2.5 Web Crippling

B3.2.5.1 Stud-to-Track Connection for C-Section Studs

When the *track* thickness is equal to or greater than the *stud* thickness, an increase in *web* crippling strength can be realized. This increased strength is attributed to the favorable synergistic effect of the *stud*-to-*track* assembly. The provisions are based on research conducted at the University of Waterloo (Fox and Schuster, 2000) and the University of Missouri-Rolla (Bolte, 2003).

Two proposed design equations were considered for adoption by Section B3.2.5.1 of the Standard for evaluating the design strength of the *stud*-to-*track connection* for *curtain wall* applications. The proposed UMR equation (Bolte, 2003) reflected the specific contribution of the screw as follows:

$$P_{nst} = P_n + \alpha P_{not} \qquad \text{(C-B3.2.5.1-1)}$$

where

α = 0.756

P_n = *Web* crippling capacity in accordance with Section G5 of AISI S100 [CSA S136] for end one-*flange* loading

P_{nst} = *Nominal strength* [*resistance*] for the *stud*-to-*track connection* when subjected to transverse loads

P_{not} = Screw pull-out capacity in accordance with Section J4.4.1 of AISI S100 [CSA S136]

The proposed equation (C-B3.2.5.1-2) was based on a formulation proposed by Fox and Schuster (2000). The design formulation for the *stud*-to-*track connection* was based on a pure *web* crippling behavior consistent with Section G5 of AISI S100 [CSA S136]. To reflect the positive contribution of the screw attachment, Fox and Schuster (2000) proposed modified *web* crippling coefficients as follows:

$$P_n = Ct^2F_y\sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad \text{(C-B3.2.5.1-2)}$$

where

P_n = *Nominal web* crippling *strength* [*resistance*] per Section G5 of AISI S100 [CSA S136] with the following coefficients:

C = *Web* crippling coefficient = 5.6

C_R = Inside bend radius coefficient = 0.01

C_N = Bearing length coefficient = 0.30

C_h = *Web* slenderness coefficient = 0.14

R = S*tud* inside bend radius

N = *Stud* bearing length

h = Depth of flat portion of *stud web* measured along plane of *web*

t = *Stud design thickness*

θ = Angle between plane of *web* and plane of bearing surface, $45° < \theta \leq 90°$

Based on the additional tests performed at UMR and the University of Waterloo, the following coefficients are recommended:

C = *Web* crippling coefficient = 3.7

C_R = *Web* slenderness coefficient = 0.19

C_N = Bearing length coefficient = 0.74

C_h = Inside bend radius coefficient = 0.019

Although there are pros and cons to each design equation, statistically they yield similar results as shown in the following:

Table B3.2.5.1-1
Comparison of Proposed Design Equations

Parameter	Waterloo Model	UMR Model
Mean	1.001	1.000
Coeff. of Variation	0.101	0.078
Ω	1.74	1.71
ϕ (LRFD)	0.88	0.90

The *safety factor* and *resistance factor* are based on assuming a member failure mode, not a *connection* failure mode.

Although both the UMR and University of Waterloo design methods will yield similar design strengths, for simplicity of design it was decided to adopt the University of Waterloo design method for the Standard. For simplicity, since $\theta = 90°$ and therefore $\sin\theta = 1$, the $\sin\theta$ term was eliminated from Equation B3.2.5.1-1.

In 2020, the maximum bearing length N is extended to 2.375 inch for single *stud* interior configuration. This is based on the test data provided in the research report by Bolte (2003).

When the *track* thickness is less than the *stud* thickness, the proposed provisions are based on the study by Fox and Schuster (2000).

The 0.5 applied to P_{nst} for locations adjacent to wall openings is based on a study by Daudet (2001).

Further research on these *stud*-to-*track connection*s was carried out in 2008 at the University of Waterloo (Lewis, 2008), with the objective being to extend the design provisions in Section B3.2.5.1 of the Standard to cover jamb *stud* members. These jamb *studs* can be single *C-shaped* members or built-up sections made from multiple members. The scope of testing was limited to the following:

1. Single *C-shaped stud* members located at the end of the *track* with two orientations of the *stud*: with the stiffening lips adjacent to the *track* end, and with the *stud web* adjacent to the *track* end.
2. Two *C-shaped studs* connected back-to-back, positioned in the *track* away from the *track* end, and positioned at the end of the *track*.
3. Two *C-shaped studs* connected toe-to-toe, positioned in the *track* away from the *track* end, and positioned at the end of the *track*.

The research showed that the *web* crippling expression in the Standard can be applied to all other combinations of single *C-shaped* and built-up members simply by changing the global *web* crippling coefficient, C. It was determined that the strength of other jamb *stud* configurations were multiples of the C=3.7 for a single *stud*. For example, a single member at the end of a *track* is 0.5C = 1.85; toe-to-toe or back-to-back members are 2C = 7.40; toe-to-toe members at the *track* end are 1.5C = 5.55; and single *stud* at the *track* end with the stiffening lips adjacent to the opening are 0.75C = 2.78. These results can only be applied within the range of the test parameters. For the *web* crippling failure mode to occur, the screws connecting the *stud*-to-*track* need to be sized according to the *stud* thickness. Provisions have been added for minimum screw sizes for the various *stud* thicknesses.

These tests were limited to combinations of *stud* and *track* where the *track* was the same thickness as the *stud*. This avoided the possibility of a *track* punch-through failure for the single *C-shaped studs* and for the toe-to-toe built-up members, but not for the back-to-back members. Consequently, a new design limit state for the *track* punch-through failure mode is proposed for the back-to-back built-up jamb members.

In situations where the back-to-back jamb *stud* configurations do not have screws connecting both *flanges* to the *track*, the *web* crippling strength [*resistance*] of the *studs* must be calculated using the provisions of AISI S100 [CSA S136]. Since the AISI S100 *web* crippling calculations for back-to-back built-up sections overestimate the *nominal strength* [*resistance*] of these jamb *studs*, the *nominal strength* [*resistance*] is calculated using twice the AISI S100 *web* crippling capacity for a single *web* member.

The *safety factors* and *resistance factors* were determined based on the test data. As a result, there are different factors for different jamb *stud* configurations.

B3.2.5.2 Deflection Track Connection for C-Shape Studs

The provisions contained in the Standard apply to a *C-shape* wall *stud* installed in a single *deflection track* application and are based on research at the Milwaukee School of Engineering (Gerloff, 2004). Based on this research, the load capacity [*resistance*] can be established by the equations in the Standard. The key parameters, as given by the equations, are illustrated by Figure C-B3.2.5.2-1.

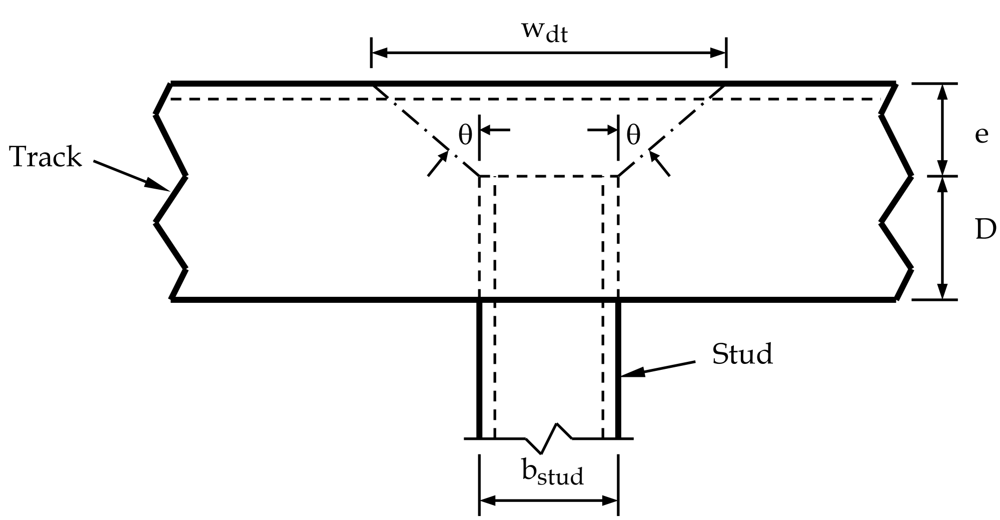


Figure C-B3.2.5.2-1 Deflection Track Connection

Because the *deflection track* detail does not provide torsional restraint for the wall *stud*, it is recommended that a line of bridging be installed near the end of the member.

For Figure C-B3.2.5.2-1, dimension "e" is selected for the sum of construction tolerances and the deflection of the floor above relative to the floor or foundation below. Dimension "D" is selected so that adequate *stud* to *track* engagement and *web* crippling bearing length remain when the floor below deflects relative to the floor above.

The background research for this provision did not include *studs* at or near terminations in the top *track*. For this case, the strength and serviceability of the *connection* may be reduced.

B3.3 Header Design

Box and back-to-back *header* beams have been commonly used in *cold-formed steel* framing. The geometry is fabricated using two *C-shaped cold-formed steel* members. Design practice for such *header* beams can be based on AISI S100 [CSA S136]. Research has determined that the application of AISI S100 [CSA S136] design provisions is conservative when *track* members are included in the assembly. This led to the development of an improved design methodology.

L-*header* beam geometries are gaining popularity in *cold-formed steel* framing. The geometry is fabricated using one or two L-shaped *cold-formed steel* members connected to a top *track* section. This geometry is commonly referred to as a single or double L-*header* because one or two angle shapes are used to create the *header*.

The *header* design provisions in the Standard do not attempt to provide a complete methodology for the design of openings in buildings. For such, the designer must consider numerous loading and serviceability issues (including out-of-plane loads) and design a complete load path consisting of members and *connections* that may include *cripple studs*, *jack studs*, *king studs*, head *tracks* and sill *tracks*. A useful reference for designers on this subject is the AISI *Cold-Formed Steel Framing Design Guide* (AISI, 2016).

B3.3.1 Back-to-Back Headers

The design methodology is based on review and analysis of the data presented in the NAHB (2003a) report *Cold-Formed Steel Back-To-Back Header Assembly Tests* (1997) and the study of Stephens (2000, 2001). The test results were evaluated and compared with the strength equations contained in the 2001 edition of the AISI Specification.

Stephens and LaBoube (2000) concluded that *web* crippling or a combination of bending and *web* crippling is the primary factor in *header* beam design for the IOF (interior one-*flange*) loading condition. Neither pure shear nor combined bending and shear were failure modes in the test program. The research study showed that using AISI S100 [CSA S136] *web* crippling equations for shapes having single *webs* for the design of box or back-to-back *header* beams gave conservative results.

B3.3.2 Box Headers

Based on studies conducted by Stephens (2001), a modification factor was derived that enables the computation of the interior one-*flange web* crippling capacity of a box *header* assembly as defined by Figure C3.4.4-2 of the Standard. The increased *web* crippling capacity is attributed to the interaction of the *track* section and the *C-shaped* section; thus, it is imperative that the *track* section be attached with the *flanges* as shown in Figure C3.4.4-2. This interaction is quantified by the ratio of *track* thickness to *C-shaped* section thickness in

Eq. B3.3.2.3-1. When computing the *web* crippling capacity for a *header* assembly, the nominal capacity computed using AISI S100 [CSA S136] is to be multiplied by 2 to reflect that there are two *webs* in the assembly. In addition to a modification to the pure *web* crippling strength, the Standard also contains an interaction equation for bending and *web* crippling of box *header* assemblies that differs from AISI S100 [CSA S136]. This interaction equation is based on the research of Stephens (2001), which included test specimens having standard perforations. Thus, the provisions of AISI S100 [CSA S136] are appropriate for *header* design.

If the top *track* section of a box *header* assembly is attached with the *flanges* up, as would be the case where the *header* beam is located directly above the opening and beneath the *cripple studs*, the provisions of Section B3.3.2.3 would not apply. *Web* crippling capacity and the combination of bending and *web* crippling should be evaluated by using Sections G5 and H3 of AISI S100 [CSA S136], and the equations for shapes having single unreinforced *webs* should be used.

The procedure to calculate the vertical deflection of a box or back-to-back *header* may be accomplished by using a composite assembly calculation, which would include the two *C-shaped* sections and the top and bottom *tracks*. However, to achieve full composite action, using this type of calculation would require an extensive (cost-prohibitive) fastener requirement between the *tracks* and the *C-shaped* sections; and therefore, it is more common to use a conservative estimate of the vertical deflection based on the full moment of inertia of the two *C-shaped* sections alone.

See Section B3.3.1, Back-to-Back Header, for additional information, as applicable.

B3.3.3 Double L-Headers

The available test data (Elhajj and LaBoube, 2000; and LaBoube, 2004) indicated that the failure mode was flexure or combination of flexure and *web* crippling. Neither pure shear nor combined bending and shear were failure modes in the test program. The tested moment capacity, M_t, was determined and compared with the computed moment capacity as defined by Section F3.1 with $F_n = F_y$ of AISI S100 [CSA S136]. The *nominal* flexural *strength* [*resistance*] was computed using the following equation:

$$M_n = S_{xc} F_y \tag{C-B3.3.3-1}$$

where

F_y = Measured *yield stress*

S_{xc} = Elastic section modulus of the effective section computed at extreme compressive stress $f = F_y$.

The section modulus of the compression *flange* was used for all computations.

It should be noted that the flexural strength is based on the section modulus of the compression *flange*; i.e., yielding of the shorter, horizontal leg of the angle. The inelastic reserve capacity of the longer, vertical leg is recognized and yielding in the extreme tensile fiber is not considered a limit state.

It should also be noted that when the design provisions of the Standard were developed, the elastic section modulus of the effective section was computed assuming that when the free edge of the element was in tension, Equations 1.1.2-3, 1.1.2-4 and 1.1.2-5 of AISI S100 [CSA S136] would apply regardless of the magnitude of h_o/b_o. Therefore, these assumptions are appropriate when calculating the elastic section modulus of the effective section using the Standard.

The L-*header* *nominal* flexural *strength* [*resistance*], M_n, can also be determined in accordance with Section F3.2 with $M_{ne} = M_y$ of AISI S100 [CSA S136]. For typical L-*header*s having a geometry as defined by the limitations of Section B3.3.3, the performance of full-scale double L-*header* beam tests (Elhajj and LaBoube, 2000; and LaBoube, 2004) has shown that the limit states of shear, *web* crippling, bending and shear, and bending and *web* crippling need not be considered when designing an L-*header* beam. This is because shear and *web* crippling failures were not indicated in any of the tests, and because a simplified conservative design approach is used. *Web* crippling is effectively prevented by the way L-*headers* are assembled. However, designers are cautioned that an L-*header* could potentially fail in shear for the combination of a very short span and a very large loading. Currently there are no limitations prescribed on minimum lengths or other factors that would prohibit shear failure in such cases. However, as a suggested procedure, shear should probably be considered when the span-to-depth ratio is less than 3.

The procedure to calculate the vertical deflection of an L-*header* is undefined because the L-*header* is an indeterminate assembly consisting of two angles, *cripple studs*, and *track* sections interconnected by self-drilling screws. However, the test results indicate that the measured assembly deflections at an applied load that equaled the *nominal load* [*specified load*] was less than L/240. Further analytical work, based on test data, would be necessary in order to develop a calculation procedure to determine the deflection of L-*header* beams.

B3.3.3.1.1 Gravity Loading

The test results summarized by Elhajj and LaBoube (2000) and LaBoube (2004) are considered to be confirmatory tests that show AISI S100 [CSA S136] Section F3.1 with $F_n = F_y$ provides an acceptable determination of the *nominal* flexural *strength* [*resistance*].

For the 10-inch (254-mm)-deep L-*header* beams having the span to vertical leg dimension, L/L_h, greater than 10, the tested *header* sections had tested moment capacities greater than the computed moment capacity defined by Eq. B3.3.3.1.1-1 in the Standard. However, for 10-inch (254-mm)-deep beams having L/L_h ratios less than 10, the tested moment capacity was on the average 10% less than the computed moment capacity (Elhajj and LaBoube, 2000). Thus, the application of Eq. B3.3.3.1.1-1 is questionable for full range of the 10-inch (254-mm) L-*headers*. A review of the data indicates that the application of Eq. B3.3.3.1.1-1 is valid for test specimens having a span to vertical leg dimension, L/L_h, of 10 or greater. For the specimens having L/L_h ratios less than 10, it is proposed that the results obtained by using Eq. B3.3.3.1.1-1 be multiplied by 0.9.

B3.3.3.1.2 Uplift Loading

A comparison of the tested to computed moment capacity ratios ranged from 0.141 to 0.309 with a mean of 0.215 (Elhajj and LaBoube, 2000). Further analysis of the tested to computed moment ratios indicated that the behavior was influenced by the ratio of L_h/t. Therefore, uplift reduction factors, R, in the Standard were developed as a function of the L_h/t ratio.

Based on the provisions of Section K2.1 of AISI S100 [CSA S136], the *safety factor* was computed to be 2.0.

B3.3.4 Single L-Headers

Prior to 2003, the Standard excluded single L-*headers*. The NAHB Research Center (2003a) study that was completed prior to 2003 tested both single and double L-*header* beams. The tests consisted of either a single-point load or a two-point load. All angles had a 1.5 inch (38.1 mm) top *flange*. The vertical leg dimensions were either 6, 8, or 10 inches (152, 203 or 254 mm). Thicknesses ranged from nominally 0.033 to 0.068 inches (0.84 to 1.73 mm). Test span lengths ranged from 36 to 192 inches (914 to 4880 mm).

An analysis of the data indicated that the behavior of the L-*header*s differed for single- versus double-angle geometries. Also, the single-point load produced test results that differed from the two-point load. Prior to 2003, there was insufficient data to develop design guidelines for single-angle L-*header*s. Thus, the data analysis did not consider data for the single-angle sections nor for the single-point loading.

In 2003, testing was completed at the NAHB Research Center (2003a) on single L-*header* beams. The tests were similar to the previously tested double L-*header* beam tests, but *header* sizes were limited to vertical leg dimensions of 6 and 8 inches (152 or 203 mm), thicknesses ranged from nominally 0.033 to 0.054 inches (0.84 to 1.37 mm), and spans were limited to 4 feet (1.219 m). From this testing, sufficient data was provided to develop design guidelines for single L-*headers* within the range of parameters tested.

LaBoube (2004), based on testing by the NAHB Research Center (2003a), demonstrated that the design methodology for double L-*headers* in the 2001 edition of the AISI *Standard for Cold-Formed Steel Framing—Header Design* was acceptable for evaluating the gravity moment capacity of single L-*headers*, within the limitations of the test program. Uplift tests on single L-*headers* were not performed as part of this test program; however, Section B3.3.4.1.2 has been reserved in the Standard for this eventuality. Further, using the provisions of Section F1 of AISI S100 of the 1996 edition of the Specification (AISI, 1996), the same Ω and ϕ factors that were prescribed in the 2001 edition of the AISI *Standard for Cold-Formed Steel Framing—Header Design* for the design of double L-*headers* would apply to single L-*headers*. As with previously tested double L-*headers*, neither pure shear nor combined bending and shear were failure modes for the tested single L-*headers*. Also, *web* crippling and combined bending and *web* crippling would be precluded from occurring because of the requirement that concentrated load applications occur at *cripple stud* locations.

B3.3.5 Inverted L-Header Assemblies

In 2005, provisions for inverted *L-headers*, as shown in Figure C-B3.3.5-1, were added to the Standard, based on rational engineering judgment, as a means to provide improved capacity for double and single *L-headers*.

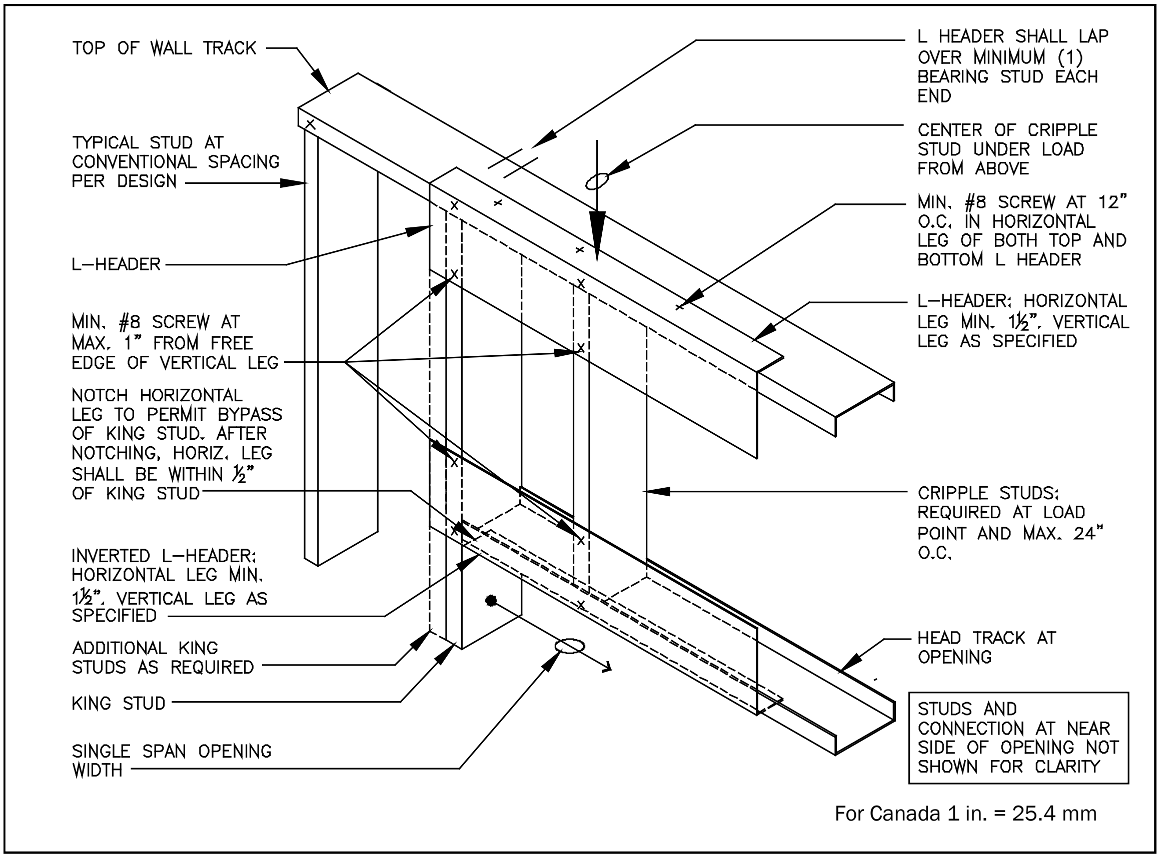


Figure C-B3.3.5-1 Inverted Single or Double L-Header Assembly (Inverted Single L-Header Shown)

B3.4 Bracing

The requirement in the Standard that each brace be designed for 2% of the design compression load [force] in the member is based on a long-standing industry practice. See further discussion in *Commentary* Section B3.2.1.

Bracing requires periodic anchorage. Bracing forces are accumulative between anchorage points.

B3.4.1 Intermediate Brace Design

Brace forces are additive, thus the Standard requires consideration of combined brace forces that when designing braces for members that experience combined loading. Design guidance is provided in AISI D110 (AISI, 2016).

B3.5 Serviceability

The Standard does not stipulate serviceability limit states. However, the *International Building Code* (ICC, 2015) sets forth deflection limits in Sections 1604.3 and 1405.10.3, and the *NFPA 5000* (NFPA, 2015) sets forth similar provisions in Section 35.1.2.8 for use in the United States and Mexico. Likewise, the *User's Guide - NBC 2010 Structural Commentaries (Part 4, of Division B)* (NRCC, 2010) sets forth deflection limits for use in Canada.

B4 Roofing Framing

B4.5 Bracing Design

The continuous bracing and *flange* bracing provisions of the Standard were deemed appropriate due to the availability of research and field experience with such assemblies. The requirements in Section B4.5(c) were adapted from AISI S100 [CSA S136] requirements for members where neither *flange* is attached to sheathing.

B5 Lateral Force-Resisting Systems

The lateral design provisions of the Standard were initially based on the requirements in the *International Building Code* (ICC, 2003) and the NFPA 5000 *Building Construction and Safety Code* (NFPA, 2003). The provisions in those codes evolved since the early work of Tarpy (1976-80), APA-The Engineered Wood Association (1993), Serrette (1995a) and the *shear wall* provisions that were first introduced into the 1997 *Uniform Building Code* (ICBO, 1997). Research conducted by Serrette at Santa Clara University and Dolan at Virginia Polytechnic Institute and State University form the technical basis for the initial design values in the Standard. Specific references to this research are cited in this *Commentary*. In 2007, provisions and design values related to *shear wall* and *strap braced wall* design, which are to be used with the 2005 *National Building Code of Canada* [*NBCC*] (NRCC, 2005), were added to the Standard based largely on research carried out under the supervision of Rogers at McGill University between 2005 and 2007. Specific references to this research are cited in the Commentary on ASI S400. At this time, the Standard does not address the Canadian design of *diaphragms*. Studies are ongoing, and it is expected that the Canadian design of these elements will be addressed in future editions of the Standard.

B5.2 Shear Wall Design

Shear walls are to be designed as either *Type I shear walls* or *Type II shear walls.*

Chord studs are typically designed to resist both tensile and compressive axial forces, and may also resist bending forces. For members resisting combined compressive axial and bending forces, AISI S100 Section H1.2 requires the determination of $\overline{P}$, $\overline{M}_x$ and $\overline{M}_y$. The determination of $\overline{P}$, $\overline{M}_x$ and $\overline{M}_y$ is specified by AISI S100 Section C1.

Where AISI S100 Section C1.1 is used, a rigorous second-order elastic analysis is required including consideration of initial imperfections and stiffness reductions. This would most commonly be accomplished via a computer model capable of capturing all of the required stiffness and load effects.

AISI S100 Section C1.2 allows first-order elastic analysis with load effects amplified using the factors B_1 to account for P-δ effects and B_2 to account for P-Δ effects. The design of *chord studs* in shear walls does not generally include end moments due to lateral translation. Therefore, the $B_2\overline{M}_{\ell t}$ term for shear wall chord studs would typically be zero. An exception could occur if a ledger *track* or similar member is fastened to the *chord stud* in such a way as to generate moments due to frame action. In this instance, these moments should be determined by analysis and the B_2 multiplier applied.

Moments due to eccentric gravity forces or other lateral forces that are not due to lateral translation of the structure are amplified by B_1. *Chord* axial forces due to gravity loads (i.e., not due to lateral translation of the structure) are not amplified in AISI S100 Eq. C1.2.1.1-2.

Chord forces due to lateral translation, $\overline{P}_{\ell t}$, are amplified by B_2.

It should be noted that B_1 and B_2 represent real increases in member forces determined on the basis of first-order elastic analysis. Design of other elements and *connections* should include the effects of this amplification. For example, the B_2 amplification of chord axial forces should be considered in the design of hold-downs and anchorage.

B5.2.1 General

Type I shear walls are fully sheathed with *steel sheet sheeting*, *wood structural panels*, gypsum board panels, or *fiberboard* panels with *hold-downs* at each end. *Type I shear walls* sheathed with *steel sheet sheathing* or *wood structural panels* are permitted to have openings where details are provided to account for force transfer around openings. Figures C-B5.2.1-1(a) and C-B5.2.1-1(b) show typical *Type I shear walls*, with and without detailing for force transfer around window openings, and with *hold-down* anchors at the ends of each wall segment. Where wall assemblies are engineered for force transfer around openings and engineering analysis shows that uplift restraint at openings is not required, the assembly may be treated as a *Type I shear wall* and *hold-downs* are required at the ends of the assembly only, as illustrated in Figure C-B5.2.1-1(b).

Type II shear walls are sheathed with *steel sheet sheeting* or *wood structural panels* with a *Type II shear wall segment* at each end. Openings are permitted to occur beyond the ends of the *Type II shear wall*; however, the width of such openings shall not be included in the length of the *Type II shear wall*. Figure C-B5.2.1-2 shows a typical *Type II shear wall.*

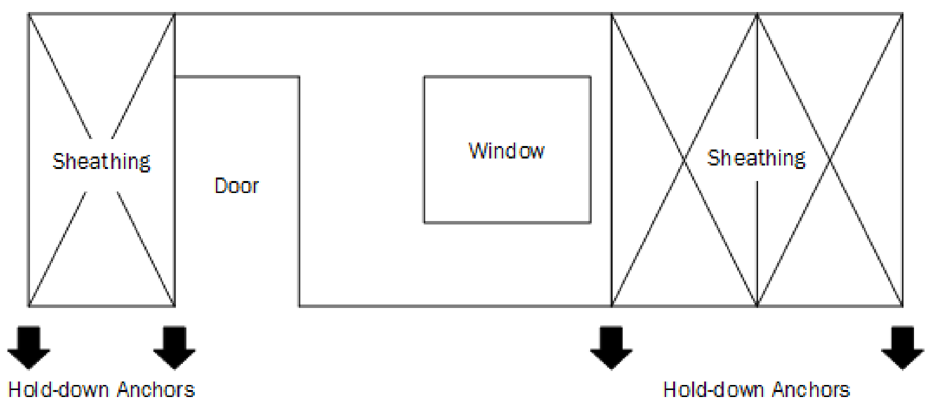


Figure C-B5.2.1-1(a) Type I Shear Walls Without Detailing for Force Transfer Around Openings

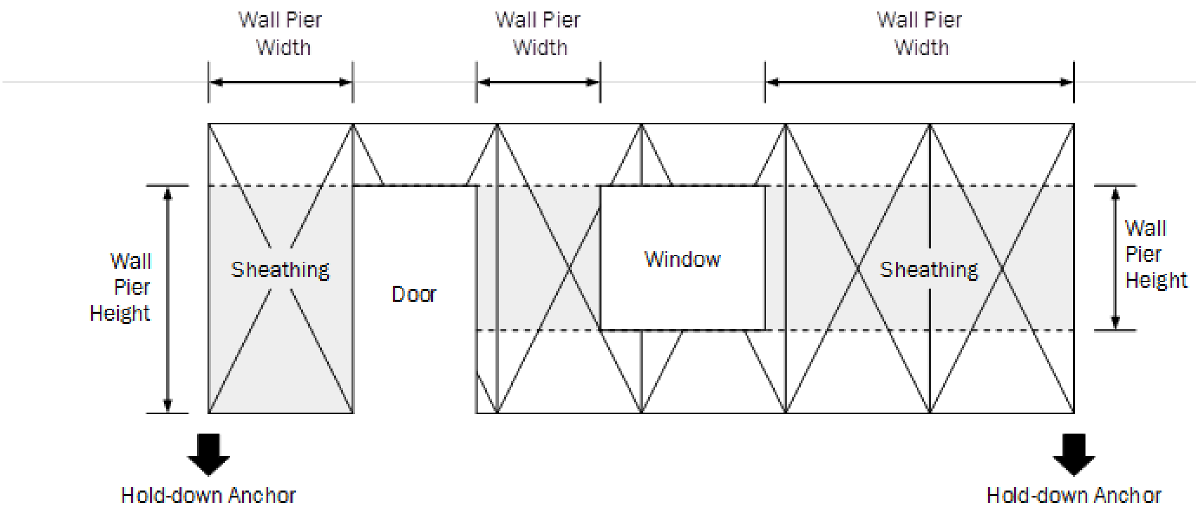


Figure C-B5.2.1-1(b) Type I Shear Wall With Detailing for Force Transfer Around Openings

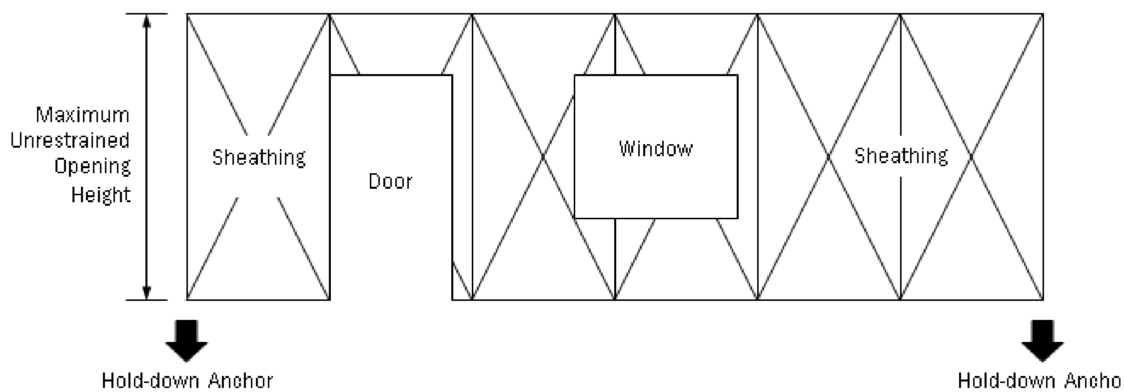


Figure C-B5.2.1-2 Typical Type II Shear Wall

B5.2.2 Nominal Strength [Resistance]

Since the tabulated values in the Standard are based on test data, it was deemed necessary to provide the user with the limiting values of the tested systems. The intent is not to prevent an engineer from using judgment, the principles of mechanics and supplemental data to develop alternate shear values from those shown in the Standard, as discussed in Section B1.2.5.2 above.

It is possible to use rational analysis to predict the *nominal shear strength* [*resistance*] of wood structural panel *shear walls* based on fastener testing. The modelling method described in Buonopane et al. (2015) has been shown to provide reasonable predictions of shear strength when compared to existing test data (Edgin, Schafer, and Madsen, 2018). The use of appropriate test-based fastener strength and stiffness data is critical to the reliability of this method.

For both *wood structural panels* and *steel sheet sheathing*, aspect ratios up to 4:1 are permitted with reductions in *nominal strength*. The reduced strength values are conservative based on 4:1 aspect ratio tests conducted by Serrette (1997).

A *shear wall* assembly using an approved adhesive to attach *shear wall* sheathing to the framing is not yet recognized by this Standard or by ASCE 7. Sufficient test data to justify acceptance of *shear walls* that use adhesive alone or in combination with fasteners to attach sheathing to the framing members was not available at the time this Standard was written. The limited existing test data indicates that *shear walls* using adhesives for sheathing attachment will generally not perform the same as *shear walls* with only fasteners attaching the sheathing to the framing.

Overdriving of the sheathing screws will result in lower strength, stiffness and ductility of a *shear wall* compared with the values obtained from testing (Rokas, 2006); hence, sheathing screws should be firmly driven into framing members but not overdriven into sheathing.

B5.2.2.1 Type I Shear Walls

In the United States and Mexico: The initial requirements for *Type I shear walls* in the Standard were based on studies by Serrette (1996, 1997 and 2002). This series of investigations included reverse cyclic and monotonic loading and led to the development of the design values and details for plywood, oriented strand board (OSB), and gypsum wall-board (GWB) *shear wall* assemblies that are included in the Standard.

In the United States and Mexico: In 2006, requirements were established for *Type I shear walls* with *fiberboard* panel sheathing based on studies by the NAHB Research Center (NAHB, 2005) and by the American Fiberboard Association (PFS, 1996 and NAHB, 2006). The *nominal strength* values for *shear walls* faced with *fiberboard* in Table B5.2.2.3-4 were based on monotonic tests of *fiberboard* sheathed, *cold-formed steel* framed *shear walls* and were compared to the tests that are the basis of the building code tabulated capacities for *fiberboard* sheathed, wood-framed *shear walls*. For the 2-inch (50.8 mm) and 3-inch (76.2-mm) edge screw spacing, the *nominal strength* values in Table B5.2.2.3-4 were based on the average peak load from tests of two 8-foot (2.438 m)-wide by 8-foot (2.428 m)-tall wall specimens. These *nominal strength* values were found to be within 90 percent of the *nominal strength* values for similarly sheathed wood-framed walls. The ratio of steel-to-wood *nominal strength* values increased as the edge (perimeter) fastener spacing increased and, therefore, extrapolating the 2/6 (92% ratio) and 3/6 (96% ratio) design values to 4/6 using a ratio of 90% was conservative. For the 4-inch (101.6 mm) edge screw spacing, the *nominal strength* values were calculated as 90 percent of the *nominal strength* value for a similarly sheathed wood-framed wall.

In the United States and Mexico: In 2011, the maximum aspect ratio of *shear wall* covered with 0.027'' *steel sheet sheathing* on single side of the *shear wall* was established based on the experiments performed at the University of North Texas (Yu, 2007). In addition, the *nominal strength* values for 0.030" and 0.033" *steel sheet sheathing* on one side were added. The addition was also based on testing at the University of North Texas (Yu, 2007; Yu et al., 2009). *Designation thicknesses* of *stud*, *track* and *blocking*, and required sheathing screw size were added to the tables as well.

In the United States and Mexico: The Effective Strip Method for determining the *nominal shear strength* for *Type I shear walls* with steel sheet sheathing is based on research by Yanagi and Yu (2014). The method assumes a sheathing strip carries the lateral load via a tension field action as illustrated in Figure C-B5.2.2.1-1. The shear strength of the shear wall is controlled by the tensile strength of the effective sheathing strip, which is determined as the lesser of the fasteners' tensile strength and the yield strength of the effective sheathing strip. The statistical analysis in Yanagi and Yu (2014) yielded an LRFD resistance factor of 0.79 for the Effective Strip Method. In order to keep consistency in resistance factors (0.65 for LRFD) specified in Standard Section B5.2.3, the original design equation in Yanagi and Yu (2014) was adjusted accordingly.

In 2020, Standard Eq. B5.2.2.3.2.1-1 was revised to be expressed with shear strength in unit length. This revision is to ensure that the predicted strength is not affected by the strength adjustment in accordance with Standard Eq. B5.2.2-2 for high aspect ratio shear walls. In addition, the upper limit of 54 mils for *stud*, *track*, and stud block was removed. This is based on the fact that the fasteners' shear strength is not affected by the upper limit in using the effective strip method.

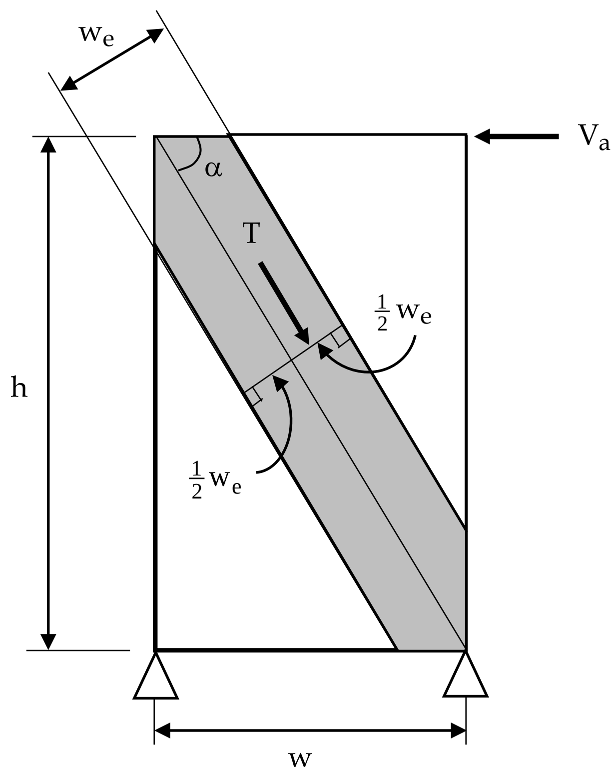


Figure C-B5.2.2.1-1 Effective Strip Method Model for Steel Sheet Sheathing

In the United States and Mexico: The *nominal strengths* were based on tests with *studs* with 1.5-inch (38 mm) x 4-inch (100 mm) punchouts at a center-to-center spacing of 24 inches (600 mm), anchor bolts with standard cut washers, and *hold-down* anchors on each end of the wall. As a result, the Standard permits the use of *studs* with standard punchouts and anchor bolts with standard cut washers, and requires *hold-down* anchors even though calculations may demonstrate that *hold-down* anchors are not necessary.

In the United States and Mexico: Factors were included, based on load duration factors given in the 2005 NDS (AFPA, 2005a) as shown in Table C-B5.2.2-1, to account for the influence of the duration of the applied load on wood strength to allow the tabulated values to be used for other in-plane lateral loads. Since the *shear wall* tests used as the basis of the Standard were carried out over a short time span, the tabulated values are for short-term duration loads (i.e., wind or seismic). However, the tabulated values for *diaphragms* were calculated using a load duration factor of 1.33, rather than the factor of 1.6 given in the 2005 NDS.

Table C-B5.2.2-1
United States and Mexico
AFPA NDS Load Duration Factors

Load Duration	Factor	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load

In Canada: *Nominal resistance* values for *steel sheet sheathed* and *wood structural panel* sheathed *shear walls* in AISI S240 match the values in AISI S400. Refer to the Commentary on AISI S400 for information on the derivation of these values. *Nominal resistance* values for gypsum and *fiberboard* sheathed *shear walls* in AISI S240 were set at 80% of the values found in Tables B5.2.2.3-3 and B5.2.2.3-4 for the United States. This reduction in resistance level is similar to what is found for the wood sheathed walls of similar construction, i.e. Table B5.2.2.3-2.

B5.2.2.2 Type II Shear Walls

The requirements for *Type II shear walls*, also known as perforated *shear walls*, in Section B5 were based on provisions in NEHRP (2000) for wood systems. In this method, the shear capacity ratio, F, or the ratio of the strength of a *shear wall* segment with openings to the strength of a fully sheathed wall segment without openings, is determined as follows:

$$F = \frac{r}{3-2r} \qquad \text{(C-B5.2.2.2-1)}$$

where

$$r = \frac{1}{1+\frac{A_0}{h\sum L_i}} \qquad \text{(C-B5.2.2.2-2)}$$

A_0 = Total area of openings
h = Height of wall
$\sum L_i$ = Sum of the length of full-height sheathing

Research by Dolan (1999, 2000a, 2000b) demonstrated that this design procedure is as valid for steel-framed systems as for all wood systems, and the IBC (ICC, 2003) and NFPA 5000 (NFPA, 2003) building codes both permit the use of *Type II shear walls* for steel systems. Test results revealed the conservative nature of predictions of capacity at all levels of monotonic and cyclic loading. The Standard does not provide a method or adjustment factor for estimating the lateral displacement of *Type II shear walls*. As such, the user should be cautious if a *Type II shear wall* is used in a deflection-sensitive design.

Table B5.2.2.2-1 in the Standard, which establishes an adjustment factor for the shear resistance, is based on the methodology described in this section and exists in essentially the same form in both the wood and steel chapters of the IBC (ICC, 2003) building code. There is also a similar table in AISI S230 (AISI, 2019); however, AISI S230 establishes an adjustment factor for the *shear wall* length rather than the *shear wall* resistance.

Although the Dolan work was based on *wood structural panel* sheathing, the Committee felt it was appropriate to extend this methodology to *shear walls* with *steel* *sheet sheathing* due to the similar performance of *wood structural panel* sheathing and *steel sheet sheathing* in monotonic and cyclic tests (Serrette, 1997) of *Type I shear walls*.

In accordance with Section B5.2.2.2 in the Standard, it is required to check the height/length ratio of each *Type II shear wall segment* and reduce the strength of each segment that has an aspect ratio greater than 2:1, but less than or equal to 4:1 by the factor of 2w/h. This aspect ratio reduction factor is cumulative with the shear resistance adjustment factor, C_a.

B5.2.3 Available Strength [Factored Resistance]

In the United States and Mexico: With the adoption of AISI S400, *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems*, the seismic requirements, including the *resistance factors*, ϕ, and *safety factors*, Ω, were eliminated from AISI S240. Seismic design using the provisions of AISI S240 are only permitted in Seismic Design Category A, or in Seismic Design Categories B or C if the Response Modification Factor, R, in accordance with ANSI/ASCE 7-10 Table 12.2-1, is equal to 3. For these conditions, where special seismic detailing is not required, the lower *resistance factor*, ϕ, and higher *safety factor*, Ω, typically required for seismic design, are not warranted.

In Canada: The *resistance factor* (ϕ) in AISI S240 matches the value in AISI S400. Refer to the Commentary on AISI S400 for information on the derivation of this value.

B5.2.5 Design Deflection

The deflection provisions are based on work performed by Serrette and Chau (2003). Equation C-B5.2.5-1 may be used to estimate the drift deflection of *cold-formed steel* light-framed *shear walls* recognized in the building codes. The equation should not be used beyond the *nominal strength* values given in the Standard. The method is based on a simple model for the behavior of *shear walls* and incorporates empirical factors to account for inelastic behavior and effective shear in the sheathing material. Specifically, the model assumes that the lateral deflection (drift) of a wall results from four basic contributions: linear elastic cantilever bending (*boundary member* contribution), linear elastic sheathing shear, a contribution for overall nonlinear effects, and a lateral contribution from anchorage/*hold-down* deformation. These four contributions are additive.

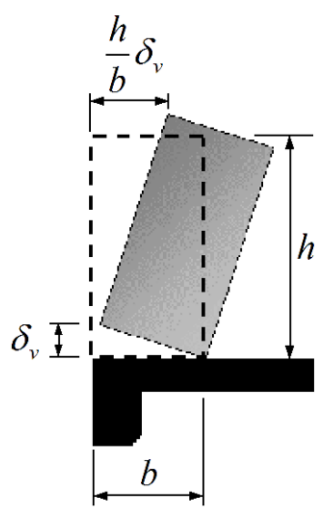


Figure C-B5.2.5-1 Lateral Contribution From Anchorage/Hold-Down Deformation

$$\delta = \frac{2vh^3}{3E_sA_cb} + \omega_1\omega_2 \frac{vh}{\rho Gt_{sheathing}} + \omega_1{}^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v \quad \text{(C-B5.2.5-1)}$$

Linear elastic cantilever bending: $\frac{2vh^3}{3E_sA_cb}$ (C-B5.2.5-2)

Linear elastic sheathing shear: $\omega_1\omega_2 \frac{vh}{\rho Gt_{sheathing}}$ (C-B5.2.5-3)

Overall nonlinear effects: $\omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2$ (C-B5.2.5-4)

Lateral contribution from anchorage/*hold-down* deformation: $\frac{h}{b}\delta_v$ (C-B5.2.5-5)

The lateral contribution from anchorage/*hold-down* deformation is dependent on the aspect ratio of the wall, as illustrated in Figure C-B5.2.5-1. The empirical factors used in the equation are based on regression and interpolation analyses of the reversed cyclic test data used in development of the *cold-formed steel shear wall* design values. The ρ term in the linear elastic sheathing shear expression attempts to account for observed differences in the response of walls with similar framing, fasteners and fastener schedules, but different sheathing material. Low values of ρ for *steel sheet sheathing* are a result of shear buckling in the sheet. The equations were based on *Type I shear walls* without openings, and the user should use caution if applying them to *Type I shear walls* with openings or to *Type II shear walls*. The *shear wall* deflection equations do not account for additional deflections that may result for other components in a structure (for example, wood sills and raised floors).

For wood structural panels, the shear modulus, G, is not a readily available value, except for Structural I plywood panels in the IBC (ICC, 2003) and UBC (ICBO, 1997) codes. However, the shear modulus may be approximated from the through thickness shear rigidity (G_vt_v), the nominal panel thickness (t) and through thickness panel grade and construction adjustment factor (C_G) provided in the AFPA Manual (AFPA, 2001). For example, G for 7/16-in. 24/16 OSB rated sheathing can be approximated as follows:

G_vt_v (24/16 span rating) = 25,000 lb/inch (strength axis parallel to framing)

t = 0.437 inch (as an approximation for t_v)

C_G = 3.1

G (approximate) = 3.1 x 25,000 / 0.437 = 177,300 psi

Thus, $C_GG_vt_v$ = 77,500 lb/inch and Gt = 77,500 lb/inch

A comparison of the $C_GG_vt_v$ and Gt values suggests that using the nominal panel thickness as an approximation to t_v is reasonable given that the deflection equation provides an estimate of drift.

Currently, the *shear wall* deflection equations do not include provisions for gypsum board or *fiberboard shear walls*. However, the reader is reminded that given the low seismic response modification coefficient, R, assigned by the building codes to gypsum board *shear walls*, it is expected that these systems will perform in the elastic range of behavior and deflections will be less likely to control the design.

In 2012, coefficients β and ρ in deflection Equation B5.2.5-1 were revised for Canadian Soft Plywood (CSP), and *steel sheet sheathing*, based on research results compiled by Cobeen (2010). CSP was differentiated from other plywoods based on the performance of that material. It should be noted that Canadian Douglas Fir Plywood (DFP) was found to behave similarly to plywood in common use in the United States.

B5.3 Strap Braced Wall Design

For *strap braced walls,* it is acceptable to compute the deflection using standard engineering analysis. Deflection calculations should consider all elements that contribute to the horizontal top of wall displacement, including axial deformation of the *studs*, elongation of the *straps*, and a lateral contribution from anchorage/*hold-down* deformation, as well as additional deflections that may result for other components in a structure (for example, wood sills and raised floors). Because loose *straps* permit lateral displacement without resistance, the Standard requires that *straps* be installed taut.

See *Commentary* Section B5.2 for discussion of *chord stud* design based on AISI S100 [CSA S136] as it applies to *shear walls*.

In addition to the considerations discussed in *Commentary* B5.2, *chord studs* in single-sided *strap braced walls* may resist strong-axis bending forces due to the eccentricity of strap forces. *Chord studs* in narrow strap-braced walls (aspect ratio ≥ 2:1) are required to consider weak-axis bending due to fixity at the strap *connections* (See Standard Section B5.3.1).

The design of *chord studs* in double-sided *strap braced walls* with aspect ratios < 2:1 does not generally include end moments due to lateral translation. Therefore, the $\overline{M}_{\ell t}$ terms in both the weak- and strong-axis for these *chord studs* would typically be zero. An exception could occur if a ledger *track* or similar member is fastened to the *chord stud* in such a way as to generate *chord* moments due to frame action. In this instance, these moments should be determined by analysis and the B_2 multiplier applied.

Where single-sided strap bracing is used, strong-axis *chord* bending forces result from the eccentricity of the strap connection to the *chord studs*. Strap forces increase due to lateral translation of the structure (P-Δ). Therefore, since the vertical component of strap force generates the *chord* strong-axis bending moment, the moment determined by first-order analysis should be amplified by B_2. In addition, the strong-axis bending in this case is perpendicular to the plane of the lateral translation of the structure. To account for P-δ effects, these strong-axis *chord* moments would also be amplified by B_1.

For narrow *strap braced walls* (aspect ratio ≥ 2:1), weak-axis *chord* moments due to joint rotational fixity result from lateral translation of the structure. Accordingly, these moments are amplified by B_2. However, the flexural stiffness due to end *connection* rotational rigidity is not considered in the analysis to contribute to the stability of the structure. Therefore, the stiffness reduction required by AISI S100 Section C1.1.1.3(b) need not be considered. See *Commentary* Section B5.3.1 for additional information regarding *chord* stud end moments due to joint rotational fixity.

B5.3.1 General

When subject to lateral force, narrow *strap braced walls* place bending demands in addition to axial demands on the *boundary elements* of the *strap braced wall*. *Strap braced walls* that have an aspect ratio (h:L) of 1:1 have insignificant bending demands; however, walls with the aspect ratio (h:L) of 2:1 have been experimentally shown to require consideration of the bending demand in the *chord studs*. Analysis indicates that the bending demands quickly increase for walls with aspect ratios greater than 1:1, and the *Standard* has chosen to require consideration of these moments for aspect ratios greater than or equal to 2:1. The *boundary elements* must be designed for the bending moments that develop in the *strap* *braced wall* as the wall undergoes displacement. Determining these bending moments requires a structural analysis where the *boundary element connections* (*stud*-to-*track*) are fully fixed. The assumption of full *joint* fixity provides a conservative approximation of the bending demand and has been shown to accurately predict observed failures in tests on strap-braced shear walls. See Mirzaei et al. (2015) for a complete discussion.

The structural analysis may be completed using frame analysis in software or in closed-form as presented here. Lateral load on a *strap braced wall* is resisted by truss action (subscript T) and frame action (subscript F). The stiffness of each in resisting lateral forces is:

$$k_T = \left[\frac{h^3}{b^2 EA_c} + \frac{(h^2+b^2)^{1.5}}{b^2 EA_s}\right]^{-1} \qquad (Eq.\ \text{C-B5.3.1-1})$$

$$k_F = \left(\frac{\dfrac{6I_b}{I_c}+\dfrac{4b}{h}}{\dfrac{6I_b}{I_c}+\dfrac{b}{h}} \times \frac{h^3}{24EI_c}\right)^{-1} \qquad (Eq.\ \text{C-B5.3.1-2})$$

where

k_T = Lateral stiffness of truss system

h = Height of wall

b = Width of wall

E = Modulus of elasticity of steel

A_c = Gross cross-sectional area of *chord stud*

A_s = Gross cross-sectional area of *strap*

k_F = Lateral stiffness of frame system

I_b = Gross moment of inertia of *track* about the axis of bending under frame action

I_c = Gross moment of inertia of *chord stud* about the axis of bending under frame action

For a shear force, V, the deflection, δ, of the wall is:

$$\delta = \frac{V}{k_F + k_T} \qquad (Eq.\ \text{C-B5.3.1-3})$$

The amount of lateral force attributed to the frame action, V_F, is:

$$V_F = (k_F)\delta \qquad (Eq.\ \text{C-B5.3.1-4})$$

V_F results in a moment at the base of the *chord stud* (M_b) and a moment above the *hold-down* (M_h) due to frame action, which can be calculated by using Equations C- B5.3.1-5 and C-B5.3.1-6:

$$M_b = \frac{V_F h}{2}\left(\frac{\dfrac{3I_b h}{I_c b}+1}{\dfrac{6I_b h}{I_c b}+1}\right) \qquad (Eq.\ \text{C-B5.3.1-5})$$

$$M_h = M_b\left(\frac{\dfrac{M_b}{0.5V_F}-h_0}{\dfrac{M_b}{0.5V_F}}\right) \qquad (Eq.\ \text{C-B5.3.1-6})$$

where h_0 is the distance from the base to the top of the *hold-down*. The assumption, consistent with experimental observations, is that the *hold-down* stiffens the *chord stud* and the critical location for axial and bending demands is at the cross-section of the *chord stud* immediately adjacent to the end of the *hold-down*. As a result, the Standard does not require that locations along the length of the *chord stud* that are stiffened by a *hold-down* or similar attachment be checked for combined axial and bending demands. This provides some relief from the large bending demands that are assumed from the assumption of full *joint* fixity.

The deflection calculated per Equation C-B5.3.1-3 is not intended to be an approximation of actual system deflection for the purposes of lateral design. The provisions for narrow *strap braced walls* are not intended to provide an increase in the *nominal strength* [*nominal resistance*] of the *strap braced wall* based on the lateral force resistance provided by frame action due to joint rigidity.

See *Commentary* Section B5.3 regarding application of the stability requirements of AISI S100 [CSA S136] Section C1 to narrow *strap-braced walls*.

B5.4 Diaphragm Design

B5.4.1 Cold-Formed Steel Sheathed Diaphragms

The Standard provides methods for determination of the *diaphragm* shear strength and shear stiffness to be determined in accordance with AISI S310 for profiled steel *diaphragm* panels commonly referred to as *steel roof deck*, non-composite *steel deck*, composite steel deck-slabs, and steel roof panels. In the U.S. and Mexico, additional *steel deck* design requirements for the design of the profiled steel panels are provided by Steel Deck Institute (SDI): ANSI/SDI RD (SDI, 2017c) for roof decks, ANSI/SDI NC (SDI, 2017b) for non-composite decks, and ANSI/SDI C (SDI, 2017a) for composite deck-slabs. In Canada, the recommended design for steel deck follows the Canadian Sheet Steel Building Institute (CSSBI): CSSBI 10M (CSSBI, 2013) for roof decks and CSSBI 12M (CSSBI, 2017) for composite decks.

Steel roof panels are profiled steel panels that act as both a structural element of the roof structure and the weather-resistant roof covering. Both the IBC in the United States and the NBC in Canada have additional requirements for roof slope, weathertightness, minimum steel thickness, and corrosion resistance that should be followed to ensure these panels are suitable as a roof covering.

This Standard leaves open the opportunity to use other types of steel sheathing for *diaphragms* that exceed the scope of profiled cold-formed steel panels through testing as specified in AISI S100 [CSA S136]. The allowance for testing provides an avenue for new and innovative products to be brought forth when testing is performed to the satisfaction of the *authority having jurisdiction* for the project.

B5.4.2 Wood Structural Panel Sheathed Diaphragms

The Standard does not currently address the design of *diaphragms* in Canada; however, pending the completion of research that is currently underway, it is expected that the design of *diaphragms* in Canada will be addressed in a future edition.

B5.4.2.1 General

The Standard permits the use of *steel sheet sheathing*, concrete or *wood structural panels* or other *approved* materials to serve as the *diaphragm* sheathing.

B5.4.2.2 Nominal Strength

The *nominal strength* of *diaphragms* is to be based upon principles of mechanics, per Section B1.2.6. Alternatively, for *diaphragms* sheathed with wood structural panels, the *nominal strength* may be determined by Section B5.4.2.2. The design values for *diaphragms* with *wood structural panel* sheathing in Table B5.4.2.2-1 were based on work by Lum (LGSEA, 1998). Lum developed *ASD* design tables using an analytical method outlined by Tissell (APA, 1993; APA 2000) for wood framing and the provisions of the 1991 NDS (AFPA, 1991). Because steel is not affected by splitting or tearing when fasteners are closely spaced, no reduction in the calculated strength was taken for closely spaced fasteners. In addition, although steel with *designation thicknesses* greater than 33 mil resulted in higher strength values, no increase in strength was included for these greater thicknesses.

It should be noted that flat *strap* used as *blocking* to transfer shear forces between sheathing panels is permitted, but is not required to be attached to framing members.

It should be noted that the *diaphragm* design values by Lum were based on the *nominal strength* of a No. 8 screw attaching *wood structural panels* to 33-mil *cold-formed steel* framing members. The 1991 NDS calculation methodology, which was used by Lum, yielded a *nominal strength* of 372 lbs and a *safety factor* of 3.3. However, the NDS methodology was revised in 2001, and the revision greatly reduced the calculated strength of screw *connection*s. Until Lum's work is updated, justification for maintaining the current *diaphragm* design values in the Standard are based, in part, on tests performed by APA (APA, 2005). Test results for single lap shear tests for a No. 8 screw attaching ½ in. plywood to 68-mil *steel sheet sheathing* indicated that the *nominal strength* [*resistance*] of the *connection* was governed by the strength of the screw in the *steel sheet sheathing*; i.e., the *wood structural panels* did not govern the capacity. Therefore, for thinner *steel sheet sheathing*, the *limit state* would likely be the tilting and bearing failure mode. For a No. 8 screw installed in 33-mil *steel sheet sheathing*, computations of *connection* capacity in accordance with AISI S100 [CSA S136] would yield a *nominal strength* of 492 lbs and a *safety factor* of 3.0. Additionally, *connection* tests for plywood attached to 33-mil *cold-formed steel* framing members were performed by Serrette (1995b) and produced an average ultimate *connection* capacity of 1177 lbs, and Serrette suggested the use of a *safety factor* of 6, as given by APA E380D. A review of the *allowable strengths*, as summarized in Table C-B5.4.2.2-1, indicates that although Lum's design values are based on an earlier edition of the NDS, the value is conservative when compared to both AISI's and Serrette's results.

Table C-B5.4.2.2-1
No. 8 Screw Shear Strength (lbs) for 33-mil Cold-Formed Steel Member

Lum		AISI 2001		Serrette	
Nominal	Allowable	Nominal	Allowable	Nominal	Allowable
372	112	492	164	1177	196

B5.4.2.4 Design Deflection

The methodology for determining the design deflection of *diaphragms* was based on a comparison of the equations used for estimating the deflection of wood frame *shear walls* and *diaphragms*, coupled with similarities in the performance of *cold-formed steel* and wood frame *shear walls*. Collectively, these comparisons suggested that the wood frame *diaphragm* equation could be adopted, with modifications to account for the difference in fastener performance, for application to *cold-formed steel light-frame construction*. The current equation for wood frame construction (ICC, 2003) is as follows:

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b} \quad \text{(C-B5.4.2.4-1)}$$

$$\text{For SI: } \Delta = \frac{0.052vL^3}{EAb} + \frac{vL}{4Gt} + \frac{Le_n}{1627} + \frac{\sum(\Delta_c X)}{2b} \quad \text{(C-B5.4.2.4-2)}$$

where

A = Area of *chord* cross-section, in square inches (mm^2)

b = *Diaphragm* width, in feet (mm)

E = Elastic modulus of *chords*, in pounds per square inch (N/mm^2)

e_n = Nail deformation, in inches (mm)

G = Modulus of rigidity of wood structural panel, in pounds per square inch (N/mm^2)

L = *Diaphragm* length, in feet (mm)

t = Effective thickness of wood structural panel for shear, in inches (mm)

v = Maximum shear due to design *loads* in the direction under consideration, in pounds per linear foot (N/mm)

Δ = The calculated deflection, in inches (mm)

$\Sigma(\Delta_c X)$ = Sum of individual *chord*-splice values on both sides of the *diaphragm*, each multiplied by its distance from the nearest support

Equations C-B5.4.2.4-1 and C-B5.4.2.4-2 apply to uniformly nailed, blocked *diaphragms* with a maximum framing spacing of 24 inches (610 mm) on center. For unblocked *diaphragms*, the deflection must be multiplied by 2.50 (APA, 2001). If not uniformly nailed, the constant 0.188 (For SI: 1/1627) in the third term must be modified accordingly.

In 2012, coefficients β and ρ in deflection Equation B5.2.5-1 were revised based on research work by Cobeen (2010). Based on *shear wall* performance, similar revisions were made to the deflection Equation B5.4.2.4-1 for the *diaphragm* systems.

B5.4.2.5 Beam Diaphragm Tests for Non-Steel Sheathed Assemblies

Consistent with wood-framed construction, for the in-plane *diaphragm*, the *nominal shear strength* [*resistance*] is permitted to be determined by a beam test in accordance with ASTM E455. The *nominal shear strength* is to be the average of a minimum of three tests in accordance with Section K2.1(a) of AISI S100 [CSA S136].

The determination of the *safety factor* and *resistance factor*, when failure occurs in the *cold-formed steel* member or sheathing-to-steel fasteners, is based on the screw *connection* statistics from Section K2.1(a) of AISI S100 [CSA S136] and is consistent with Section I6.3.1 of AISI S100 [CSA S136].

The *safety factor* of 2.8 and *resistance factor* of 0.6 are used to be consistent with the wood sheathing failure as prescribed by the wood industry (AFPA, 2008).

The beam test provides diaphragm performance for an essentially rectangular building configuration. The aspect ratio of 4:1 reflects the limit for essentially rectangular buildings as prescribed by AISI S230 (AISI, 2019).

C. INSTALLATION

C2 Material Condition

To ensure structural performance is in compliance with the engineered design, *structural members*, *connectors*, *hold-downs* and mechanical fasteners must not be damaged. Damage assessment is not within the purview of this Standard. The *design professional* should be consulted when damage alters the cross-section geometry of a *structural member*, or damages to *connectors*, *hold-downs* and mechanical fasteners beyond the specified tolerances.

C2.1 Web Holes

AISI S100 [CSA S136] stipulates design requirements for members with standard *web* holes. In the field, these "*web* holes" may also be referred to as "*punchouts,*" "utility holes," "perforations" and "*web* penetrations." In *structural members*, *web* holes are typically 1.5 in. (38 mm) wide × 4 in. (102 mm) long and are located on the center line of the *web*. The *web* holes are generally spaced 24 in. (610 mm) on center.

C2.2 Cutting and Patching

This Standard places restrictions on acceptable methods for cutting of framing members so that cut edges are not excessively rough or uneven and protective metallic coatings are not damaged in areas away from cut edges. It is noted that shearing includes a variety of mechanical methods including, but not limited to, the use of hydraulic shears and hole punches during manufacturing; and portable hydraulic shears, hand-held electric shears, aviation snips, and hole punches during fabrication and installation.

Coping, cutting or notching of *flanges* and *edge stiffeners* is not permitted for *structural members* without an *approved* design. For guidance on design for coped members in *trusses,* refer to Section E4.6.2.

C2.3 Splicing

Structural members may be spliced; however, splicing of *studs* and *joists* is not a common practice and is not recommended. If a *structural member* requires splicing, the splice *connection* must be installed in accordance with an *approved* design.

C3 Structural Framing

C3.1 Foundation

An uneven foundation may cause problems. The specified ¼ in. (6.4 mm) gap has been deemed acceptable industry practice.

C3.2 Ground Contact

To minimize the potential for corrosion, care must be taken to avoid direct contact between the *cold-formed steel* framing and the ground. In addition to direct contact, it is important to minimize the potential for corrosion resulting from ambient moisture. The *applicable building code* is cited as the authoritative document that will provide guidance concerning minimum separation distances from the ground to the framing member, installation requirements for moisture barriers, and the necessary ventilation of the space.

C3.3 Floors

To avoid premature failure at a support and to achieve in-line framing, full bearing of the *joist* on its supporting wall is necessary. The intent of specifying that the *track* and *joist webs* are not to be in direct contact with each other is to prevent floors from creating an unwanted noise (e.g., squeaks).

C3.4 Walls

C3.4.1 Straightness, Plumbness and Levelness

Wall *studs* must be installed plumb to avoid the potential for secondary bending moments in the member. In 2015, a reference to tolerances in ASTM C1007 (ASTM, 2011d) for plumbness and levelness was added.

C3.4.3 Stud-to-Track Connection

The *stud* must be nested and properly seated into the top and bottom *track* to provide for adequate transfer of forces and minimize axial deflections. Each *flange* of the *stud* should also be attached to the *tracks* to brace the top and bottom of the *stud* against weak axis and torsional displacements.

The maximum end gap specified by this Standard is based on traditional industry practice. The gap specified in Section C3.4.3(a) is only for axial load bearing walls, which is defined in Section 202 of the IBC (ICC, 2012) as any metal or wood stud wall that supports more than 100 pounds per linear foot (1459 N/m) of vertical load in addition to its own weight.

Axial loads in a wall *stud* in excess of the capacity of the screw *connection* between the *stud* and its seating tracks will be transferred between the *stud* and *track* in bearing. In this situation, if a *stud* is not properly seated, relative movement between the *stud* and the *track* may occur, which reduces or closes any gap between the end of the *stud* and the *track*.

To determine the influence of this relative movement between the *stud* and the *track* on the serviceability of sheathed wall assemblies, a testing program was conducted at the University of Missouri-Rolla (Findlay, 2005). The UMR test program only considered the *stud* and the *track* having the same thickness. For thinner *stud* and *track* materials (0.054 inches (1.37 mm) or less), testing showed that relative movement between the *stud* and the *track* was accommodated through a combination of *track* deformation and screw tilting. In these cases, the *connection* remained intact and was capable of resisting uplift forces and preventing *stud* weak axis and torsional displacement. For thicker materials (greater than 0.054 inches), testing showed that the relative movement between the *stud* and the *track* could result in shear failure of the screws. In these cases, testing indicated a smaller end gap tolerance (e.g., 1/16 inch (1.59 mm)) would be desirable to limit relative movement and potential screw failure.

In addition to a smaller specified end gap to avoid potential screw failure in *track* thicker than 0.054 inches (1.37 mm), a smaller specified gap may also be desirable for multi-story structures where the accumulation of gap closures may become significant. Special considerations may also be desirable for heavily loaded *cold-formed steel* structures and conditions susceptible to deflections.

One method to help achieve adequate end bearing and minimize undesirable relative movement between the *stud* and the *track* is to specify a smaller 1/16 in. (1.59 mm) gap. This is a relatively simple criterion to verify. However, not exceeding the 1/16 in.

(1.59 mm) gap is on occasion difficult to achieve, particularly with 0.097 in. (2.46 mm) or thicker *track*.

Another method to achieve adequate end bearing is to pre-compress the *stud* inside the *track*. This is often accomplished when panelizing the *stud* walls and pre-compressing the wall panel before connecting the *track* to the *studs*. Wall panelization and pre-compression, particularly for multi-story *cold-formed steel* construction, has become a common practice in several regions of North America. With pre-compression, it can be relatively assured that the *stud* will be seated in the *track*, regardless of the gap after pre-compression. At present, there are no specific guidelines for the amount of pre-compressive force required to assure proper seating. Industry practice has been to compress the wall panel until the *studs* visually seat inside the radius of the *track* and before the *studs* begin to buckle. Jacking force will vary depending upon the *stud* size and wall panel height, but is typically several hundred pounds minimum per *stud*. Guidelines for verification of proper pre-compression are nonexistent, but typically pre-compression will result in gaps of 1/16 in. (1.59 mm) or less for the majority of *stud*-to-*track connections*, with no gap exceeding 1/8 inch.

A third method to help ensure adequate end bearing is to oversize the *web* depth of the *track* to minimize any gap. *Track* oversized 1/16 to 1/8 in. (1.59 mm to 3.18 mm) will usually have a flat *web* between the *track* radii that is greater than the depth of the *stud*, enabling the *stud* to bear directly on the *web* instead of the *track* radii. This method will often be desirable when bearing walls are stick-framed instead of panelized and pre-compressed. One disadvantage is the oversized *track* may make a flat wall finish more difficult.

For all thickness of materials, testing has shown that the gap between the sheathing and the floor should be equal to or greater than the gap between the *stud* and the *track*.

C3.5 Roofs and Ceilings

Proper installation and alignment of *roof joists* and *ceiling joists* is necessary to ensure the proper load transfer from the *rafter*/*ceiling joist connection* to the wall framing. To avoid premature failure at a support and to achieve in-line framing, full bearing of the *joist* on its supporting wall framing member or a minimum 1-1/2 in. (38 mm) end bearing is necessary.

C3.6 Lateral Force-Resisting Systems

Proper installation, consistent with the construction of the tabulated systems that were tested, is deemed necessary for the performance of the system. Flat *strap* used as *blocking* to transfer shear forces between sheathing panels is permitted, but is not required to be attached to framing members. For *wood structural panel*, gypsum board and *fiberboard* sheathing, screws must be installed through the sheathing to the *blocking*. Sheathing screws should be driven to the proper depth appropriate for the head style used. Bugle, wafer and flat head screws should be driven flush with the surface of the sheathing; pan head, round head, and hex-washer head screws should be driven with the bottom of the head flush with the sheathing. See *Commentary* Section B5.2.2 for more discussion on overdriving sheathing screws.

C4 Connections

C4.1 Screw Connections

C4.1.2 Installation

Screw length must be adequate to ensure that screw fasteners extend through the steel *connection* a minimum of three (3) exposed threads, as illustrated in Figure C-C4.1.2-1.

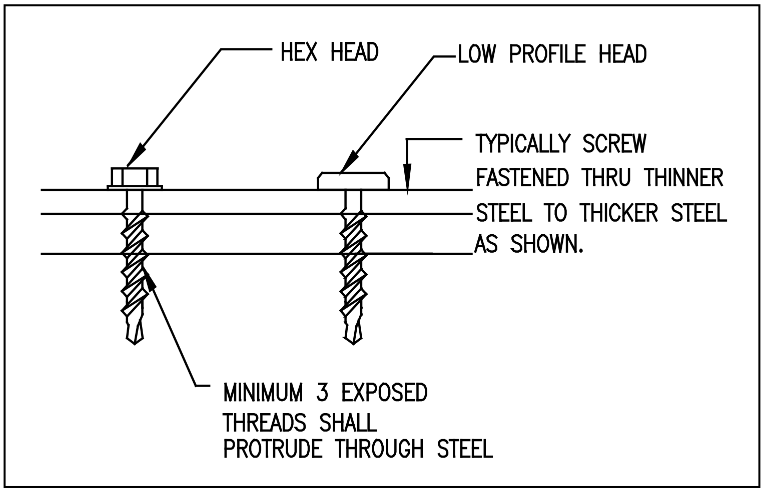


Figure C-C4.1.2-1 Screw Grip Range

C4.1.3 Stripped Screws

It is unreasonable to expect that there will be no stripped screws in a *connection*. Research at the University of Missouri-Rolla (Sokol et al., 1999) has shown that the structural performance of a single-shear screw *connection* is not compromised if screws in the *connection* have been inadvertently stripped during installation. This research serves as the basis for the requirements of the Standard.

C4.1.4 Overdriven Screws in Shear Walls and Diaphragms Sheathed With Wood Structural Panels

Overdriven screws have been shown to have a detrimental effect on the capacity of *shear walls* and *diaphragms* sheathed with *wood structural panels*. Although overdriven screw fasteners are unlikely when installed by trained framers using properly adjusted tools, screw fasteners are sometimes overdriven and the attendant capacity reduction is addressed in this section. The criteria of strength reduction were developed based on typical changes in stiffness and strength that occur in overdriven screws as reported in Vieira and Schafer (2012).

C4.2 Welded Connections

In 2020, the Standard recognized that weld areas located within the building envelope and shielded from direct contact with moisture from the ground or the exterior climate do not require treatment as galvanic concerns are minimal and sufficient protection is provided by the surrounding zinc coating. When not located within the building envelope or not shielded from direct contact with moisture from the ground or the exterior climate, the Standard establishes a minimum requirement that the weld area must be treated with a corrosion-resistant coating, such as a zinc-rich paint. In addition, interior atmospheric conditions such as indoor swimming pools and freezers, as well as abnormal conditions such as coastal areas (CFSEI, 2007b) might warrant treatment of welds.

C5 Miscellaneous

C5.1 Utilities

C5.1.1 Holes

The design should include references to pre-punch hole sizes or limitations to accommodate electrical, telecommunication, plumbing and mechanical systems. Field-cut holes are generally discouraged, but are not uncommon. Field-cut holes, if necessary, are required to comply with Section C2.1. There are several methods whereby holes can be cut in the field, such as a hole-punch, hole saws, and plasma cutters.

Holes that penetrate an assembly containing steel framing that has a fire resistance rating will need to be designed with through-penetration firestop systems. The acceptance of this fire resistance design is based on the *applicable building code*.

C5.1.2 Plumbing

Direct contact with copper piping should be avoided in order to prevent galvanic action from occurring. Methods for preventing the contact from occurring may be through the use of nonconductive and noncorrosive grommets at *web* penetrations or through the use of nonmetallic brackets (a.k.a. isolators) fastened to hold the dissimilar metal building products (e.g., piping) away from the steel framing. Plastic pipe does not require protection if it is in contact with the *cold-formed steel* framing member, but consideration should be made for the installation of nonmetallic brackets to hold the pipe away from the hole in the steel in order to prevent noise and prevent the steel from cutting into the pipe.

C5.1.3 Electrical

Nonmetallic sheathed wiring must be separated from the *cold-formed steel* framing member in order to comply with the *National Electrical Code* (NFPA, 2011). Contained within the *National Electrical Code* is a provision that requires nonmetallic sheathed cable to "be protected by bushings or grommets securely fastened in the opening prior to the installation of the cable." Cable following the length of a framing member will need to be secured (e.g., supported) at set lengths; for this purpose, small holes in the *web* may be beneficial for the attachment of tie-downs (e.g., nylon cable ties, nylon zipper ties, etc.). When installing wiring or cables within a framing member (e.g., through or parallel to member), the intent of the *National Electrical Code* further requires that the wiring or cables be located 1-1/4 inches (32 mm) from the edge of the framing member. When 2-1/2-inch (64-mm)-wide wall *studs* are used, the restrictions concerning edge clearance apply.

C5.2 Insulation

The cavity insulation must be installed such that the width of the insulation extends from the face of the *web* of one framing member to the face of the *web* of the next framing member. In the case of *cold-formed steel* framing, designs should specify "full-width" insulation in order to differentiate the insulation that is normally supplied (nominal width).

To enhance the thermal performance of *cold-formed steel* framed construction, board insulation (such as continuous insulation or insulating sheathing) may be used in conjunction with cavity insulation. Guidance on the use of board and batt insulation is given in *Thermal*

Design and Code Compliance for Cold-Formed Steel Walls (SFA, 2008). Designs should also take into consideration the effects of moisture when assessing the application of both cavity and continuous insulation, in this case dew point. The ASHRAE *Handbook of Fundamentals* contains information useful for determining dew point (ASHRAE, 2009).

D. QUALITY CONTROL AND QUALITY ASSURANCE

D1 General

D1.1 Scope and Limits of Applicability

Chapter D provides minimum requirements for *quality control* and *quality assurance* for material control and installation for *cold-formed steel light-frame construction*. Minimum observation and *inspection* tasks deemed necessary to ensure quality *cold-formed steel light-frame construction* are specified.

Chapter D does not apply to *cold-formed steel nonstructural members*. Chapter D does not apply to the manufacture of *cold-formed steel structural members*, *connectors* or *hold-downs* other than material control, nor to manufacture of mechanical fasteners or welding consumables. Chapter D does not address *quality control* or *quality assurance* for other materials and methods of construction.

In Chapter D, material control refers to the general oversight of the materials by the *component manufacturer* and *installer* and involves procedures for storage, release and movement of materials. Throughout the manufacturing and construction processes, including the associated *inspections*, materials are identified and protected from degradation. Additionally, nonconforming items are identified and segregated as needed.

The Chapter D requirements are patterned after similar requirements developed by the American Institute of Steel Construction for structural steel and the Steel Deck Institute for *steel deck*.

Chapter D defines a comprehensive system of *quality control* requirements on the part of the *component manufacturer* and *installer* and similar requirements for *quality assurance* on the part of the project representatives of the *owner* when such is deemed necessary to complement the *contractor's quality control* function. These requirements exemplify recognized principles of developing involvement of all levels of management and the workforce in the *quality control* process as the most effective method of achieving quality in the constructed product. Chapter D supplements these *quality control* requirements with *quality assurance* responsibilities as are deemed suitable for a specific task.

AISI S202, *Code of Standard Practice for Cold-Formed Steel Structural Framing*, indicates that the *component manufacturer* or *installer* is to implement a *quality control* system as part of their normal operations. The *registered design professional* should evaluate what is already a part of the *component manufacturer's* or *installer's quality control* system in determining the *quality assurance* needs for each project. Where the *component manufacturer's* or *installer's quality control* system is considered adequate for the project, including compliance with any specific project needs, the "special inspection" or *quality assurance* plan may be modified to reflect this. Similarly, where additional needs are identified, supplementary requirements should be specified.

The terminology adopted for use in Chapter D is intended to provide a clear distinction of *component manufacturer* and *installer* requirements and the requirements of others. The definitions of *quality control* and *quality assurance* used here are consistent with usage in related industries. It is recognized that these definitions are not the only definitions in use.

For the purposes of this Standard, *quality control* includes those tasks performed by the *component manufacturer* and *installer* that have an effect on quality or are performed to measure or confirm quality. *Quality assurance* tasks performed by organizations other than the *component manufacturer* and *installer* are intended to provide a level of assurance that the product meets the project requirements. The terms *quality control* and *quality assurance* are used throughout Chapter D to describe *inspection* tasks required to be performed by the *component manufacturer* and *installer* and representatives of the *owner,* respectively. The *quality assurance* tasks are *inspections* often performed when required by the *applicable building code* or *authority having jurisdiction*, and designated as "special inspections," or as otherwise required by the project *owner* or *registered design professional*.

D1.2 Responsibilities

The requirements in Chapter D are considered adequate and effective for most *cold-formed steel light-frame construction*.

Where the *applicable building code* and *authority having jurisdiction* require the use of a *quality assurance* program, Chapter D outlines the minimum requirements deemed effective to provide satisfactory results in *cold-formed steel light-frame construction*. There may be cases where supplemental *inspections* are advisable. Additionally, where the *contractor's quality control program* has demonstrated the capability to perform some tasks that this plan has assigned to *quality assurance*, modification of the plan could be considered.

D2 Quality Control Programs

Many quality requirements are common from project to project. Many of the processes used to produce *cold-formed steel light-frame construction* have an effect on quality and are fundamental and integral to the *component manufacturer's* or *installer's* success. Consistency in imposing quality requirements between projects facilitates more efficient procedures for both. The *construction documents* referred to in Chapter D are, of necessity, the versions of the *plans*, *specifications*, and *approved shop drawings* and *approved installation erection drawings* that have been released for construction, as defined in AISI S202. When responses to requests for information and change orders exist that modify the *construction documents*, these are also part of the *construction documents*. When a building information model is used on the project, it is also a part of the *construction documents*.

Elements of a *quality control* program can include a variety of documentation such as policies, internal qualification requirements, and methods of tracking production progress. Any procedure that is not apparent subsequent to the performance of the work should be considered important enough to be part of the written procedures. Any documents and procedures made available to the *quality assurance inspector* should be considered proprietary and not distributed inappropriately.

The *inspection* documentation should include the following information:

(1) The product inspected
(2) The process that was conducted
(3) The name of the inspector and the time period within which the *inspection* was conducted
(4) Non-conformances and corrections implemented

Records can include marks on pieces, notes on drawings, process paperwork, or electronic files. A record showing adherence to a sampling plan for pre-welding compliance during a given time period may be sufficient for pre-welding observation *inspection*.

The level of detail recorded should result in confidence that the product is in compliance with the requirements.

In 2020, provisions were added to Section D2.1 of the Standard helping to better coordinate its requirements with the current model building code; i.e., the *International Building Code* (IBC) (ICC, 2018). Section 1703 of the IBC provides requirements for an "approved agency," which is a defined term in the IBC. Section 2211.1.3.3 of the IBC builds on those provisions by stating: "Trusses not part of a manufacturing process that provides requirements for quality control done under the supervision of a third-party quality control agency in accordance with AISI S240 Chapter D shall be fabricated in compliance with Sections 1704.2.5 and 1705.2, as applicable." Stated another way, where the component manufacturer engages with an approved third-party quality control agency for supervision of its quality control procedures, the submittal of quality control documents required in Section D3.1 and performance of quality assurance inspection required in Section D6.4.1; i.e., the in-plant quality assurance inspections, are not required. This language was added to Section D2.1. Note that while the current language in the IBC is specific to trusses, due to the similarity in manufacturing processes, it is reasonable to extend these provisions to other component assemblies, such as wall panels. Other components manufacturers still need to engage with a quality control agency approved by the authority having jurisdiction.

D3 Quality Control Documents

D3.1 Documents to be Submitted

The documents listed must be submitted so that the *registered design professional* can evaluate that the items prepared by the *component manufacturer* or *installer* meet the *registered design professional's* design intent. This is usually done through the submittal of *shop drawings* and *installation drawings*. In many cases digital building models are produced in order to develop *shop drawings* and *installation drawings*. In lieu of submitting *shop drawings* and *installation drawings*, the digital building model can be submitted and reviewed by the *registered design professional* for compliance with the design intent. For additional information concerning this process, refer to the AISC *Code of Standard Practice for Steel Buildings and Bridges* Appendix A, Digital Building Product Models.

The exception in Section D3.1.1.3 limits the additional requirements in Sections D3.1.1.1 and D3.1.1.2 to *lateral force-resisting systems* with relatively high capacities.

D3.2 Available Documents

The documents listed must be made available for review by the *registered design professional*. In some instances, the *registered design professional* may require the submittal of additional documents. Certain items are of a nature that submittal of substantial volumes of documentation is not practical, and therefore it is acceptable to have these documents reviewed at the *component manufacturer's* or *installer's* facility by the *registered design professional* or *quality assurance* agency. Additional commentary on some of the documentation listed in this section is as follows:

(a) Documents related to *connectors*, *hold-downs* and mechanical fasteners are required only in cases where *connectors*, *hold-downs* and mechanical fasteners are being installed.

(b) Documents related to welding are required only in cases where welding of *cold-formed steel structural members* is being performed.

(c) Specific welders may not be known until installation begins; therefore, welding personnel performance qualification records may not be made available until immediately before installation begins.

(d) Because the selection and proper use of welding filler metals is critical to achieving the necessary levels of strength and quality, the availability for review of welding filler metal documentation and welding procedure specifications (WPSs) is required. This allows a thorough review on the part of the *registered design professional*, and allows the *registered design professional* to have outside consultants review these documents, if needed.

(e) The *component manufacturer* and *installer* maintain written records of welding personnel qualification testing. Such records should contain information regarding date of testing, process, welding procedure specifications, test plate, position, and the results of the testing. In order to verify the six-month limitation on welder qualification, the *component manufacturer* and *installer* should also maintain a record documenting the dates that each welder has used a particular welding process.

D5 Inspection Personnel

D5.1 Quality Control Inspector

The *component manufacturer* or *installer* determines the qualifications, training and experience required for personnel conducting the specified *inspections*. Qualifications should be based on the actual work to be performed and should be incorporated into the *component manufacturer's* or *installer's quality control* program. *Inspection* of welding should be performed by an individual who, by training and/or experience in metals fabrication, *inspection* and testing, is competent to perform *inspection* of the work. Recognized certification programs are a method of demonstrating some qualifications, but they are not the only method nor are they required by Chapter D for *quality control* inspectors.

D5.2 Quality Assurance Inspector

The *quality assurance* agency determines the qualifications, training and experience required for personnel conducting the specified *quality assurance inspections*. This may be based on the actual work to be performed on any particular project. Qualification for the *quality assurance* inspector may include experience, knowledge and physical requirements. These qualification requirements are documented in the *quality assurance* agency's written practice. AWS B5.1 (AWS, 2003) is a resource for qualifications of a welding inspector.

D6 Inspection Tasks

D6.1 General

Chapter D defines two *inspection* levels for required *inspection* tasks and labels them as either "observe" or "perform." This is in contrast to common building code terminology which uses or has used the terms "periodic" or "continuous." However, this is consistent with the AISC and SDI standards that were used as a pattern for this Standard.

The tables in Sections D6.5 through D6.10 list the required *inspection* tasks for *quality control* and *quality assurance*. If *inspections* identify nonconforming material or workmanship, the need for additional *inspections* and rejection of material is to be assessed in accordance with Section D7.

The tables in Sections D6.5 through D6.10 also list the required documentation tasks for *quality assurance*. Documentation tasks for *quality control* are not required by Chapter D, but should be as defined by the applicable *quality control program* of the *component manufacturer* or *installer*.

D6.2 Quality Control Inspection Tasks

Quality control documentation is an internal record for the *component manufacturer* or *installer* to record that the work has been performed and that the work is in accordance with the *shop drawings* or *construction documents*, as applicable. Depending upon the *component manufacturer's* or *installer's quality control program*, the method of documentation may vary.

Model building codes, such as the *International Building Code* (ICC, 2018), often make specific statements about performing *inspections* to *approved construction documents*. AISI S202 requires the transfer of information from the *contract documents* into accurate and complete *shop drawings* and *installation drawings*. Therefore, relevant items in the *plans* and *specifications* that must be followed in component manufacturing and installation should be placed on the *shop drawings* and *installation drawings*, or in typical notes issued for the project. Because of this provision, *quality control inspection* may be performed using *shop drawings* and *installation drawings*, not the original *plans* and *specifications*.

D6.3 Basic Frame Inspection Tasks

This section is to differentiate *inspection* tasks intended to be performed by the *authority having jurisdiction* from *inspection* tasks intended to be performed by the *quality assurance inspector*, and to clarify when these *inspection* tasks are required.

D6.4 Quality Assurance Inspection Tasks

Model building codes, such as the *International Building Code* (ICC, 2018), make specific statements about performing *inspections* to *approved construction documents*. Accordingly, the *quality assurance inspector* should perform *inspections* using the original *plans* and *specifications*. The *quality assurance inspector* may also use the *shop drawings* and *installation drawings* to assist in the *inspection* process.

D6.5 Coordinated Inspection

Coordination of *inspection* tasks may be needed for *component manufacturers* in remote locations or distant from the project itself, or for *installers* with projects in locations where *inspection* by a local firm or individual may not be feasible or where tasks are redundant. The approval of both the *authority having jurisdiction* and *registered design professional* is required for *quality assurance* to rely upon *quality control*, so there must be a level of assurance provided by the quality activities that are accepted.

In 2020, provisions were added to Section D6.5.1 of the Standard to coordinate its requirements with the current model building code; i.e., the *International Building Code* (IBC) (ICC, 2018). Section 1704.2.5 of the IBC waives the requirement for third-party "special inspection" where the work is done on the premises of an "approved fabricator," which is a defined term in the IBC. While the current language in the IBC is specific to fabricators, due to the similarity in processes, it is reasonable to extend these provisions to component manufacturers. The component manufacturer still needs to be approved by the authority having jurisdiction.

D6.6 Material Verification

Compliance of *cold-formed steel structural members* typically includes verification of product identification in accordance with Section A5.5 and shape dimensions and sizes in accordance with an *approved* design or *approved* design standard. The manufacturer of *cold-formed steel structural members* is typically responsible for verification of material in accordance with Section A3, corrosion protection in accordance with Section A4, and *base steel thickness* in accordance with Section A5.1.

The *installer* should consider Section G1.1 of AISI S202 when establishing material control procedures for *cold-formed steel structural members*. *Cold–formed steel structural members* that lack product identification are typically tested to determine conformity.

D6.7 Inspection of Welding

Compliance of welds typically includes verification of weld size, length and location.

D6.8 Inspection of Mechanical Fastening

Compliance of mechanical fasteners typically includes verification of mechanical fastener type, diameter, length, quantity, spacing, edge distance and location.

D6.9 Inspection of Cold-Formed Steel Light-Frame Construction

Compliance of cold-formed steel light-frame construction typically includes verification of component assembly, structural member and connector sizes and locations; bracing, blocking and bearing stiffener sizes and locations; and bearing lengths.

D6.10 Additional Requirements for Lateral Force-Resisting Systems

Compliance of *cold-formed steel lateral force-resisting system* installation typically includes verification of *shear walls*; diagonal *strap bracing* and *gusset plates*; *hold-downs* and anchor bolts; *collectors* (drag struts); and *diaphragms*.

The exception in Section D6.10 limits the additional requirements to *lateral force-resisting systems* with relatively high capacities.

In Table D6.10-2, a welder identification system is a system maintained by the *component manufacturer* or *installer*, as applicable, by which a welder who has welded a joint or member can be identified.

E. TRUSSES

E1 General

E1.1 Scope and Limits of Applicability

Cold-formed steel trusses are planar structural components. Structural performance depends on the *trusses* being installed vertically, in-plane, and at specific spacing, and being properly fabricated and braced. The Standard describes the materials used in a *cold-formed steel truss*, as well as design, fabrication, and bracing procedures for *truss* members.

This Standard is intended to serve as a supplement to AISI S100 [CSA S136]. The provisions provided in Chapter E are also intended to be used in conjunction with the other chapters of the Standard.

E2 Truss Responsibilities

The Standard adopts Section I1 of AISI S202 for the responsibilities of the individuals and organizations involved in the design, fabrication and installation of *cold-formed steel trusses*. Alternate provisions as agreed upon by the involved parties are permitted.

E3 Loading

The Standard does not establish the appropriate loading requirements for which a *truss* should be designed. In most cases, these loads are adequately covered by the *applicable building code* or standard.

E4 Truss Design

The provisions contained in this section of the Standard address the various design aspects related to *truss* strength [*resistance*]. The strength [*resistance*] determinations required by the Standard are in accordance with either the *Allowable Strength Design* (*ASD*), *Load and Resistance Factor Design* (*LRFD*) or *Limit States Design* (*LSD*) methods given by AISI S100 [CSA S136], except where additional research studies have indicated an alternative approach is warranted.

E4.3 Analysis

The structural analysis requirements contained in the Standard are based on available information pertaining to the behavior of *cold-formed steel C-shaped* section *truss* assemblies (Harper, 1995; LaBoube and Yu, 1998). These requirements do not preclude the use of more rigorous analysis or design assumptions as determined by rational analysis or testing.

E4.4 Member Design

E4.4.1 Properties of Sections

AISI S100 [CSA S136] has been shown to be highly reliable for determining the design cross-section properties of *C-shapes* and other simple geometries. For more complex shapes, such as those utilizing longitudinal stiffeners, AISI S100 [CSA S136] Direct Strength Method design provisions may be used to estimate the load-carrying capacity. Tests in accordance with Section K2.1 of AISI S100 [CSA S136] can also be used.

E4.4.2 Compression Chord Members

When subjected to gravity load, the compression *chord member* may experience the combined effects of bending and axial compression. The design for combined load effects is permitted to use the equations developed through research or in accordance with Section H1.2 of AISI S100 [CSA S136].

Engineering design specifications recognize the need for using rational analysis or test to define an effective length factor. The Standard permits the use of either rational analysis or testing.

Based on research on *C-shaped* section *trusses* conducted at the University of Missouri-Rolla (UMR) (Harper, 1995; Ibrahim, 1998), it was determined that the unbraced lengths, L_x and L_t, may be taken as equal to the distance between the panel points. It was also discovered that where structural sheathing is attached to the *chord member* and where the compression *chord member* is continuous over at least one intermediate panel point, and is continuous from the *heel* to the *pitch break*, *heel* to *heel* (in the case of a parallel chord *truss*), or breakpoint of a *truss*, L_y may be taken as the distance between sheathing connectors. Engineering judgment indicates that where sheathing is not attached to the top *chord member,* L_y may be taken as the distance between panel points.

In the *cold-formed steel truss* industry, the sheathing is used as a structural component and the *connection* spacing of the sheathing to the *cold-formed steel* member is a design consideration. Therefore, sheathing connector spacing is indeed a structural requirement, and that is why the spacing of the connector is used when designing the *truss* chords for compression when there is structural sheathing applied.

The UMR research also determined that for a structurally sheathed *C-shaped* section *truss* where the compression *chord member* is continuous over at least one intermediate panel point, and is continuous from the *heel* to the *pitch break*, or breakpoint of a *truss,* K_x, K_y, and K_t may be taken as 0.75. For other compression *chord members*, based on engineering judgment, K_x, K_y, and K_t should be taken as unity.

An alternative design assumption for *chord members* in compression, based on engineering practice and judgment, is to assume that the effective length be taken as the distance between two adjacent points of contraflexure. In such case, the effective length factor and C_m should be taken as unity.

The required effective length factors and unbraced lengths given in the Standard for hat-shapes are based on engineering judgment. The *Z-shape* requirements are based on proprietary testing.

Consistent with AISI S100 [CSA S136], the end moment coefficient, C_m, should be taken as 0.85, unless a more rigorous analysis is performed to justify another value.

Requirements in the Standard for the evaluation of the bending strength [*resistance*] are based on engineering judgment.

Ibrahim et al. (1998) determined that when a *C-shaped* section compression *chord member* is subject to concentrated load at a panel point, the interaction of axial compression, bending and *web* crippling must be considered. The researchers proposed the following *ASD* interaction equation:

$$\frac{P}{P_{no}} + \frac{M}{M_{n\ell o}} + \frac{R}{R_n} \leq \frac{1.49}{\Omega} \qquad (Eq.\ \text{C-E4.4.2-1})$$

where

P = Compression axial load

M = Bending moment

R = Concentrated load

P_{no} = *Nominal* axial *strength* [*resistance*] computed at $f = F_y$

$M_{n\ell o}$ = *Nominal* flexural *strength* [*resistance*] computed at $f = F_y$

R_n = *Nominal* interior one-*flange web* crippling *strength* [*resistance*]

Ω = *Safety factor*

= 1.95

The values of P and M are to be determined by structural analysis for the panel point in question, where R is the applied concentrated load at the panel point. The *nominal strengths* [*resistances*] are to be computed using AISI S100 [CSA S136]. Based on a statistical analysis consistent with load and resistance factor design, the *safety factor* was determined. The Standard provides an equation applicable to the *ASD*, *LRFD* and *LSD* methods.

E4.4.3 Tension Chord Members

The design requirements prescribed by the Standard for tension *chord members* are based on experience and engineering judgment.

E4.4.4 Compression Web Members

The behavior of a compression *web member* is a function of the *connection* of the *web member* to the *chord member*. For example, a common *connection* detail of *C-shaped chord* and *web members* is to attach the respective members back-to-back through their *webs*. Such a *connection* detail creates an eccentric loading condition in the *web member*. When an axial load is applied to a *truss web member* in this type of *truss* construction, this eccentric loading condition will produce a bending moment in the member that is acting out-of-plane to the *truss*. This bending moment needs to be analyzed using Section E4.4.4 of this Standard. In addition to the check in this Standard, a compression *web member* is to be analyzed with the axial load alone using Chapter E of AISI S100 [CSA S136] with consideration of flexural buckling and local buckling.

Researchers at the University of Missouri-Rolla (Rieman, 1996; Ibrahim et al., 1998) determined that for a *C-shaped* compression *web member* that is attached through its *web* element, the interaction of axial compression and out-of-plane bending may be determined by the following *ASD* interaction equation:

$$\frac{\Omega_c RP}{P_n} + \frac{\Omega_b C_{my} RPe}{M_{ny}\alpha_y} \le 1.0 \qquad \text{(C-E4.4.4-1)}$$

where

$$R = -\left(\frac{L/r}{173}\right)^2 + \frac{L/r}{88} - 0.22 \ge 0.6 \qquad \text{(C-E4.4.4-2)}$$

L = Unbraced length of the compression *web member*

r = Radius of gyration of the full section about the minor axis

P_n = *Nominal axial strength* [*resistance*] based on Sections E2 and E3 of AISI S100 [CSA S136]. Only flexural and local buckling need to be considered.

e = Eccentricity of compression force with respect to the centroid of the full section of the *web member*

Other variables are defined in Section C5.2.1 of the 1996 edition of the AISI

Specification (AISI, 1996).

The parameter, R, is an experimentally determined reduction imposed on the axial load. The equation is a fit to the average test data, which is a common practice in *cold-formed steel* research. To recognize the lower limit on the tested L/r ratio, the Standard stipulates $R \geq 0.6$. The intent of R is to recognize the increased significance of the bending effect, compared to the axial effect for longer length *web members*. Unique to the application of the interaction equation is the determination of the *nominal axial strength* [*resistance*] based on flexural buckling alone. Research showed that the minor axis bending, which resulted from the eccentrically applied axial load, created a member deflection that enabled only flexural buckling. Thus, the behavior of the *web member* was determined predominately by bending resulting from the eccentric load. The parameters P, Ω_b, Ω_c, C_{my}, M_{ny} and α_y are defined in accordance with Section C5.2.1 of 1996 edition of the Specification (AISI, 1996). The Standard provides an equation applicable to the *ASD*, *LRFD* and *LSD* methods.

For compression *web member* cross-sections other than a *C-shape* attached through its *web* element, which has symmetry of loading, the axial compression load may be taken as acting through the centroid of the section.

When computing the *available strength* [*factored resistance*], the effective lengths, K_xL_x, K_yL_y and K_tL_t, may be taken as the distance between the centers of the member's end *connection* patterns. This assumption is consistent with the analysis approach used by UMR researchers (Rieman, 1996; Ibrahim et al., 1998).

E4.4.5 Tension Web Members

Tension *web members* may experience a reduction in load-carrying capacity when subjected to combined axial load and bending. For *C-shaped* sections, this may be attributed to the dominant behavior being that of bending resulting from the eccentric load. However, testing has not documented that the combined loading compromises the integrity of the tension member. Therefore, for a tension *web member* connected to the *web* element of a *chord member*, or connected to a *gusset plate*, the Standard permits the axial tension load to be taken as acting through the centroid of the *web member's* cross-section.

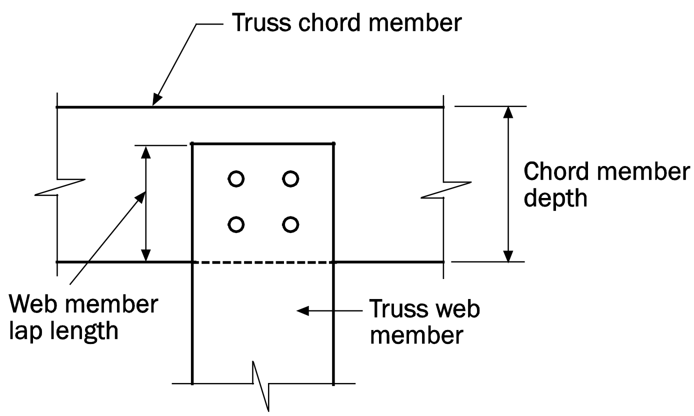
E4.4.6 Eccentricity in Joints

The Standard does not specify the use of a multiple or single node structural analysis model to account for the effects of eccentricity in joints. The *truss* stiffness will differ based on whether a multiple or single node analysis is performed. When a multiple node analysis is used, a node should be placed at each *web member* location where the center line of the *web member* meets the center line of the *chord member*. When performing a single node analysis, additional design considerations may be necessary. For example, eccentricity created by the spatial relationship of the *web members* and the *chord member* at a joint may generate additional moments, shears, and/or axial forces. Such moments and forces may be directly reflected in a multiple node analysis model. Thus, when using a single node analysis model, a secondary analysis and design check of the joint, or a load test may be required to justify the design.

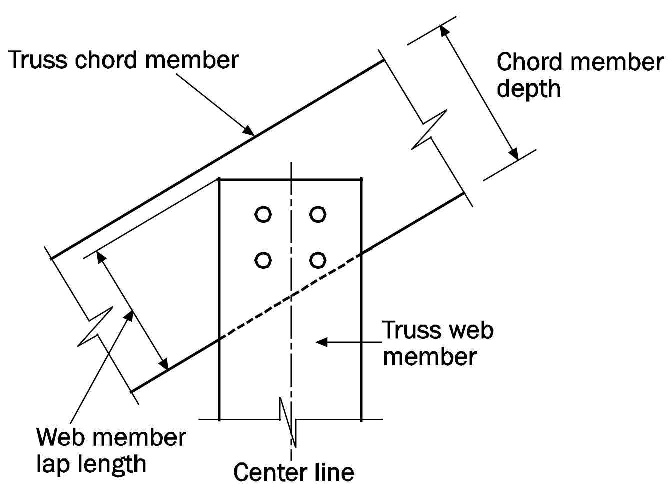
The Standard defines a *web member* lap length as 75% of the *chord member* depth. This minimum lap length is assumed, based on engineering judgment, to serve as a *web* shear stiffener for the *chord member*. The *chord member* segment between the assumed stiffeners is to be investigated for combined bending and shear, where a stiffened shear panel is assumed, in accordance with Equation H2-2 of AISI S100 [CSA S136]. For *truss* configurations having the *web member* lap length less than 75% of the *chord member's* depth, the *chord member* is to be investigated for combined bending and shear in accordance with Equation H2-1 of AISI S100 [CSA S136]. Refer to Figures C-E4.4.6(a) and C-E4.4.6(b) for a pictorial definition of the term "*web member* lap length" for two configurations of a *truss web member* and *truss chord member connection*. Rational design assumptions for this "*web member* lap length" must be used when other *connection* geometries are encountered.

Along the length of the *chord member*, at the mid-point between the intersecting *web members*, shear is to be evaluated by Section G2 of AISI S100 [CSA S136]. The shear buckling coefficient is taken to be consistent with the assumed shear panel condition at the segments' ends as defined by Section G2.3 of AISI S100 [CSA S136].

Based on experience, where screws are used as the connector, a minimum of four screws should be used in a *web member* to *chord member connection* and the screws should be equally distributed in their group.



(a) Web Member Lap Length for Flat Truss Chord



(b) Web Member Lap Length for Sloped Truss Chord

Figure C-E4.4.6 Web Member Lap Lengths

E4.5 Gusset Plate Design

To establish a design methodology for thin *gusset plate connections* in compression, a testing program consisting of 49 specimens was conducted at the University of Missouri-Rolla (Lutz, 2004). Two separate models were developed to predict the capacity of the plates. Both plate buckling and column buckling models were studied. Although both models are sufficient for calculating the strength [*resistance*] of the *gusset plates*, it was recommended that the plate-buckling model be used in design. The plate-buckling model, assuming $f=F_y$, $k=4$ and $w=W_{min}$, provided a better correlation to the test data. A limited number of tests were performed to determine the strength gain in *gusset plates* with edge stiffeners. The results of tests in which both edges of the *gusset plate* parallel to the applied load had edge stiffeners showed an approximate strength increase of 25% for the plates.

The *gusset plate* design provisions in the Standard require that W_{min} be taken as the lesser of the actual *gusset plate* width or the Whitmore section, which defines a theoretically effective cross-section based on a spread-out angle of 30° along both sides of the *connection*, beginning at the first row of fasteners in the *connection*. The first row of fasteners is defined as the row of fasteners that is the furthest away from the section of *gusset plate* being considered. Figure C-E4.5-1 illustrates how W_{min} can be determined for a typical fastener pattern connecting a *truss chord member* to a *gusset plate* at a typical *pitch break connection* at the ridge of a roof *truss*. Determining W_{min} for other conditions would be analogous.

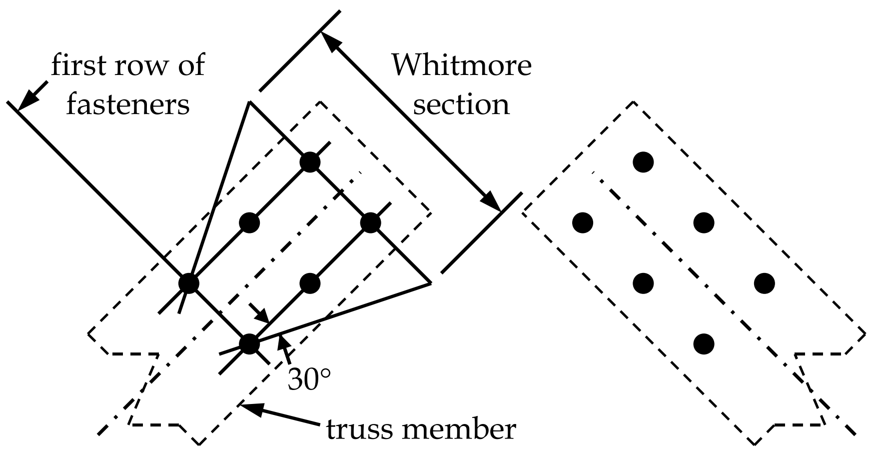
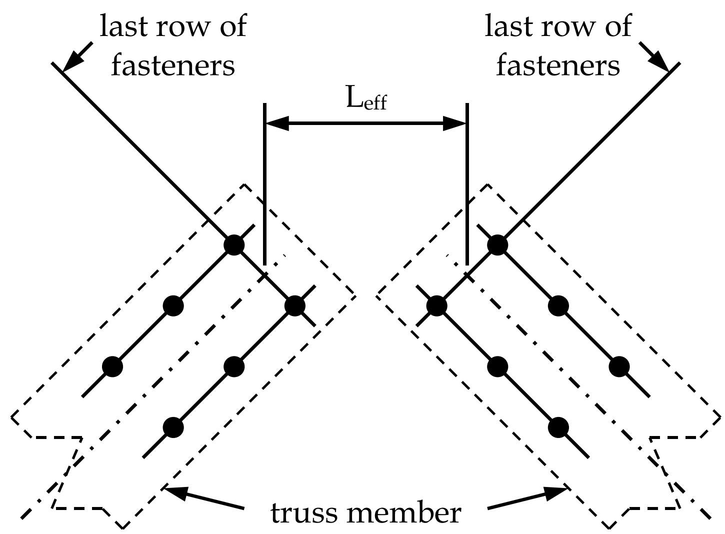


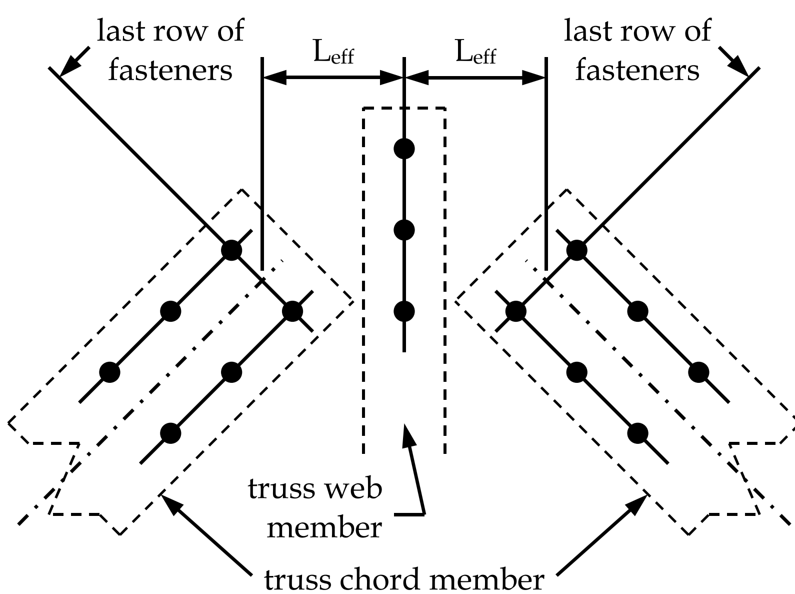
Figure C-E4.5-1 Whitmore Plate Width

The *gusset plate* design provisions in the Standard require that L_{eff} be taken as the average length between the last rows of fasteners of adjacent *truss* members. Figure C-E4.5-2 illustrates how L_{eff} can be determined for a typical *pitch break connection* at the ridge of a roof *truss*. Determining L_{eff} for other conditions would be analogous.

For *gusset plates* in tension, reference is made to the requirements of AISI S100 [CSA S136]. These requirements include checks on the gross and net areas of the *gusset plate*, shear lag and group or tear-out of fasteners. Engineering judgment is required to determine the portion of the *gusset plate* to be included in the gross and net area checks.



(a) Without King Post



(b) With King Post

Figure C-E4.5-2 Effective Length for Typical Pitch Break Connection

E4.6 Connection Design

E4.6.1 Fastening Methods

Although the common fastening system used by the industry is the self-drilling screw, the Standard permits the use of bolts, welds, rivets, clinches, and other technologies as *approved* by the *truss designer*. Screw, bolt, and weld *connections* are to be designed in accordance with AISI S100 [CSA S136]. If other fastener types, such as rivets, clinches, rosettes, adhesives, etc., are to be used in the fabrication of the *truss*, the design values are to be determined by tests, and the *available strength* [*factored resistance*] determined in accordance with Section K2.1 of AISI S100 [CSA S136].

For the design of connecting elements, such as plates, *gusset plates*, and brackets, reference is made to AISI S100 [CSA S136], which in turn makes reference to Section J4 of AISC 360 (AISC, 2010).

E4.6.2 Coped Connections for C-Shaped Sections

The design engineer should give special attention to the *heel* and *pitch break connections* of the *truss* to ensure structural integrity of the *truss*.

At a *pitch break*, coped members may be reinforced to prevent *web* buckling of the *chord member*. Attachment of a *track* section of the same thickness as the *chord member*, thus creating a box section, and having a length equal to the depth of the *chord member* has been shown to provide adequate reinforcement (Ibrahim, 1998). Lateral *bracing* is also important to stabilize the *pitch break* from overall buckling. At the *heel*, a *bearing stiffener* may be needed to preclude *web* crippling (Koka, 1997).

At a *heel connection*, UMR research (Koka, 1997) determined that coping reduces both the shear buckling and *web* crippling strength [*resistance*] of the coped bottom *chord member*. The UMR research proposed that where a coped *flange* had a *bearing stiffener* with a minimum moment of inertia (I_{min}) of 0.161 $in.^4$ (67,000 mm^4), the shear strength [resistance] could be calculated in accordance with AISI S100 [CSA S136] Section G2, but required a reduction as defined by the following factor, R:

$$R = 0.976 + \frac{0.556c}{h} - \frac{0.532d_c}{h} \le 1.0 \qquad \text{(C-E4.6.2-1)}$$

The cited limits in the Standard reflect the scope of the experimental study and apply only to *connections* where the bottom *chord member* is coped.

Where a *bearing stiffener* not having the minimum moment of inertia is used, *web* crippling controlled the *heel connection* strength [*resistance*] (Koka, 1997). Therefore, the Standard requires that the computed end one-*flange* loading *web* crippling strength [*resistance*] at the *heel*, as determined by AISI S100 [CSA S136] Section G5 be reduced by the following factor:

$$R = 1.036 + \frac{0.668c}{h} - \frac{0.0505d_c}{h} \le 1.0 \qquad \text{(C-E4.6.2-2)}$$

The cited limits in the Standard reflect the scope of the experimental study.

Where c = length of cope and d_c = depth of cope as illustrated in Figure C-E4.6.2-1. I_{min} of the stiffener is computed with respect to an axis parallel to the *web* of the bottom *chord member*.

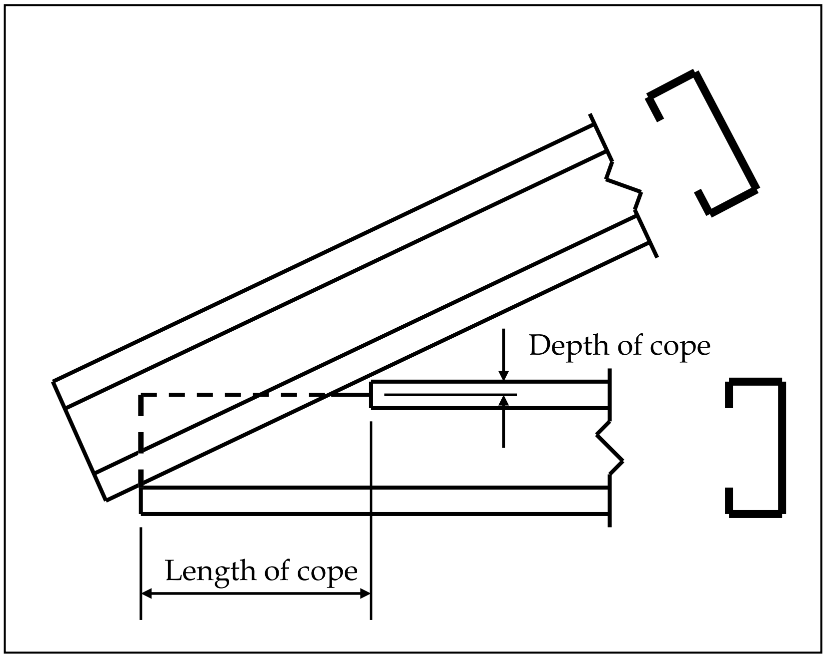


Figure C-E4.6.2-1 Definition of Truss Coping Dimensions

E4.7 Serviceability

Serviceability limits are to be chosen based on the intended function of the structure, and should be evaluated based on realistic loads and load combinations as determined by the *building designer*. Because serviceability limits depend on the function of the structure and the perception of the occupant, it is not possible to specify general limits in the Standard. As a guide to the designer, the maximum allowable deflection of the *chord member* of a *truss* resulting from gravity load, excluding dead load, may be taken as the following:

(a) Span/360 for plaster ceilings
(b) Span/240 for flexible type ceilings
(c) Span/180 for no finished ceiling
(d) Span/480 for floor systems

Although the use of a deflection limit has been used to preclude vibration problems in the past, some floor systems may require explicit consideration of the dynamic characteristics of the floor system.

Truss serviceability is evaluated at *nominal* [*specified*] load. When computing *truss* deflections, the Standard permits the use of the full cross-sectional area of the *truss* members. The use of full areas is warranted because a *truss* system is a highly indeterminate structural system, and local buckling of an individual member does not appreciably affect the stiffness of the *truss* at design load.

E5 Quality Criteria for Steel Trusses

The practices defined herein have been adopted by the Standard as commonly accepted practice. In the absence of other instructions in the contract documents, the provisions of Section E4 are the quality standard for the manufacturing processes of steel *trusses* to be used in conjunction with an in-plant quality assurance procedure and a *truss* design.

E6 Truss Installation

Cold-formed steel trusses are planar structural components. The structural performance depends on the *trusses* being installed vertically, in-plane, at specified spacing, and being properly braced. The *installer* is responsible for receipt, storage, erection, installation, field assembly, and *bracing*. The practices defined herein have been adopted by the Standard as commonly accepted practice.

A maximum bottom chord permanent lateral restraint and brace spacing of 10 feet is suggested, based on field experience and limited testing.

E6.1.1 Straightness

The *truss* installation tolerances defined in Section E6.1.1 have been used for many years in the prefabricated *truss* industries of both *cold-formed steel* and wood with good success. The tolerances listed in this section are *truss* assembly tolerances and not individual member tolerances. Member tolerances are outlined in the Standard. *Cold-formed steel trusses* are typically used with structural sheathing applied to the top chord. This sheathing is designed to support lateral loads and act as a diaphragm. This diaphragm system behavior for *trusses* with the structural sheathing is what also enables the adoption of a seemingly more liberal out-of-straightness.

E6.1.2 Plumbness

The *truss* installation tolerances defined in Section E6.1.2 have been used for many years in the prefabricated *truss* industries of both *cold-formed steel* and wood *trusses*.

E7 Test-Based Design

Design calculations require the application of *approved* materials and cross-section properties. When calculations are used to define the structural performance of a *truss* assembly, the structural performance may be verified by full-scale test. When the structural performance cannot be determined by calculation, the structural performance must be determined by test. AISI S921 provides guidance for full-scale load tests.

F. TESTING

In 2015, Chapter F was created to allow reference to applicable AISI S900-series test standards.

In 2020, methods for testing *truss* components and assemblies, formerly in S240 Appendix 2, were incorporated into AISI S921.

F1 General

Section F1 lists all the AISI S900-series test standards that are deemed to be generally applicable to *cold-formed steel* structural framing applications.

APPENDIX 1, CONTINUOUSLY BRACED DESIGN FOR DISTORTIONAL BUCKLING RESISTANCE

Calculation of the nominal distortional buckling strength in flexure and in compression in accordance with AISI S100 [CSA S136] may utilize the beneficial system effect of sheathing fastened to the compression *flange(s)* of *floor joists*, *ceiling joists*, and *roof rafters* through the calculation of the rotational stiffness provided to the member, k_ϕ. It is also permissible, and indeed conservative, to ignore this beneficial restraint and assume k_ϕ=0.

Testing was conducted to determine the provided rotational restraint in typical framing systems (Schafer et al. 2007, 2008). From this testing it was determined that the sheathing rotational restraint could be divided into two parts: one from the sheathing and one from the connector. For the sheathing industry, standard values as provided by the APA, *Panel Design Specification* (APA, 2004) and the Gypsum Association (GA, 2001) are provided. As reported in Schafer et al. (2007, 2008), the gypsum board provided stiffness in-line with or exceeding reported values but the deformation capacity was severely limited; therefore, it was determined that gypsum board should only be relied upon in a serviceability check, not for strength limit states. Section L2 of AISI S100 [CSA S136] provides a serviceability methodology for distortional bucking; therefore, the design method specifically references Appendix 1 for checking serviceability with gypsum board. For the connector, the experimentally determined values are fit to a simple empirical expression and then provided in tabular form. The testing was conducted with fasteners at 12 in. (305 mm) o.c.; therefore, the Standard requires that this fastener spacing (or less) be maintained.

Design examples and design aids for the application of these methods to floor joists are provided in CFSEI Technical Note G100-08 (Schafer, 2008).

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1. Advisory Note: The Light Gauge Steel Engineers Association (LGSEA) in 2006 changed its name to the Cold-Formed Steel Engineers Association (CFSEI). CFSEI is located at AISI headquarters in Washington, DC.

American Iron and Steel Institute

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Washington, DC 20001
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